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Final Report-IITRI Project M-254

STRUCTURAL MATERIALS
FOR HARDENED PERSONNEL SHELTERS

by
John A. Havers

This report was prepared for the Office of Civil Defense, Department of Defense, under Work Order OCD-OS-62-66, Research Subtask 1151-A.

December 1963

IIT RESEARCH INSTITUTE
(Formerly Armour Research Foundation)
Technology Center
Chicago 16, Illinois

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for

Office of Civil Defense
Department of Defense
The Pentagon
Washington 25, D.C.

December 1963

PREFACE

This is the final report on ARF Project M-254, "Structural Materials for Hardened Personnel Shelters." This study was sponsored by the Office of Civil Defense, Department of Defense, Washington, D.C., under Contract No. OCD-OS-62-66. Mr. N. A. Meador, O.C.D., functioned as monitor for the contract. The work was performed during the period of April 1962 to June 1963.

Several members of the IIT Research Institute staff have contributed to this study. Mr. J. D. Stevenson has prepared the design examples contained in Chapter 4, and Mr. J. Lukes has provided computer programs for many of the design tabulations. Mr. W. Truesdale performed the experimental phases of the soil studies described in Appendix B, and Appendix A was prepared by Mr. R. Barnett.

Respectfully submitted,
IIT RESEARCH INSTITUTE

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ABSTRACT

Structural materials are evaluated for use in fully-buried personnel shelters, located above the ground water table. Analytical relationships are supplied for the structural elements of these shelters, assuming shelter configurations of a rectangular cubicle, a horizontal 180° arch and full cylinder, and a 180° dome and full sphere. In-place costs are derived for suitable structural materials, and cost equations are supplied for estimating the in-place cost of structural elements.

Various structural units are used in preparing alternative designs for a 100-man capacity shelter in the 10 psi to 200 psi overpressure region. Cost trends are indicated, and minimum structural costs are related to design level of overpressure.

Summary

of

Research Report

STRUCTURAL MATERIALS FOR HARDENED PERSONNEL SHELTERS

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J. A. Havers

December 1963

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OCD-OS-62-66

Subtask 1151-A

IIT RESEARCH INSTITUTE
Technology Center
Chicago 16, Illinois

SCOPE OF WORK OF CONTRACT

"The Contractor, in consultation and cooperation with the Government, shall furnish the necessary facilities, personnel and other services as may be required to conduct materials research for shelter structures as specifically provided for herein and generally consistent with the outline of work contained in Contractor's proposal No. 62-373 K dated 20 December 1961. The specific work and services hereunder shall include, but not necessarily be limited to, the following:

(1) investigate materials needed to produce a broader choice of shelter construction materials and techniques for the fabrication of underground shelters to achieve lower costs or take advantage of a wider range of industrial capabilities in a large-volume program

(2) investigate the material properties, production capabilities and probable costs of shelter structural materials, such as corrugated steel plates, steel structural shapes, pre-cast concrete, reinforced concrete, fiberglass and plastics as applied to standard configurations for underground group shelters; and

(3) investigate novel possibilities such as chemical stabilization of soil, rammed earth and adobe."

APPROACH

During initial discussions between the contractor and the Office of Civil Defense it was agreed that the personnel shelters under study would be considered as fully buried, buried at shallow depths, and located above the permanent ground water level. It was further agreed that the external design environment for these shelters would be compatible with weapon yields of one to 100 megatons, producing side-on surface overpressures of 10 psi to 200 psi at the shelter locations. A shelter capacity of 100-men was selected for study purposes.

It was intially postulated that structural materials for buried shelters could best be compared on the basis of their estimated in-place structural costs* under a range of service conditions. Several representative shelter configurations (rectangular cubicle, horizontal cylinder, horizontal 180° arch, 180° hemi-spherical dome and sphere) were accordingly analyzed in terms of their basic structural elements for various possible framing systems. The material parameters of these elements were then varied, over a range of simulated loading conditions, to determine the least-cost structural designs for each element. After assuming stated dimensional and layout criteria, these least-cost structural elements were then assembled into alternative shelter combinations, and the composite cost of each shelter subsequently calculated. By performing repetitive trials, a "least in-place structural cost" relationship for the 100-man capacity shelter was developed as a function of overpressure within the 10 psi to 200 psi design range. This relationship is shown graphically on Figure S-1, where the structural cost and optimum configuration of the least-structural-cost 100-man shelter are indicated as functions of design overpressure.

The report is divided into five major chapters, with a bibliography supplied as a sixth chapter. The first chapter describes the research approach used in the study, including the several simplifications employed in the dynamic analysis of the structural elements, and also summarizes the major limitations of the study. The second chapter supplies estimates of in-place unit costs and projections of future availability for the major structural materials. The third chapter, the most lengthy in the report, contains derivations for the many analytical equations required for study of the structural elements. Extensive design tables are supplied, as well as generalized cost equations for each structural element. In particular cases, minimum-cost solutions for the generalized cost equations are provided.

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* "In-place" structural cost, as used in this study, is based upon average material costs in the Chicago Metropolitain Area during 1963. To the basic material cost is added fabrication costs and any charges for transportation and erection. To this cumulative total is added an additional 40 percent to provide for general overhead, job overhead, and contractors profit. The costs of site acquisition and preparation, Government supervision, A&E fees, etc., are not included.

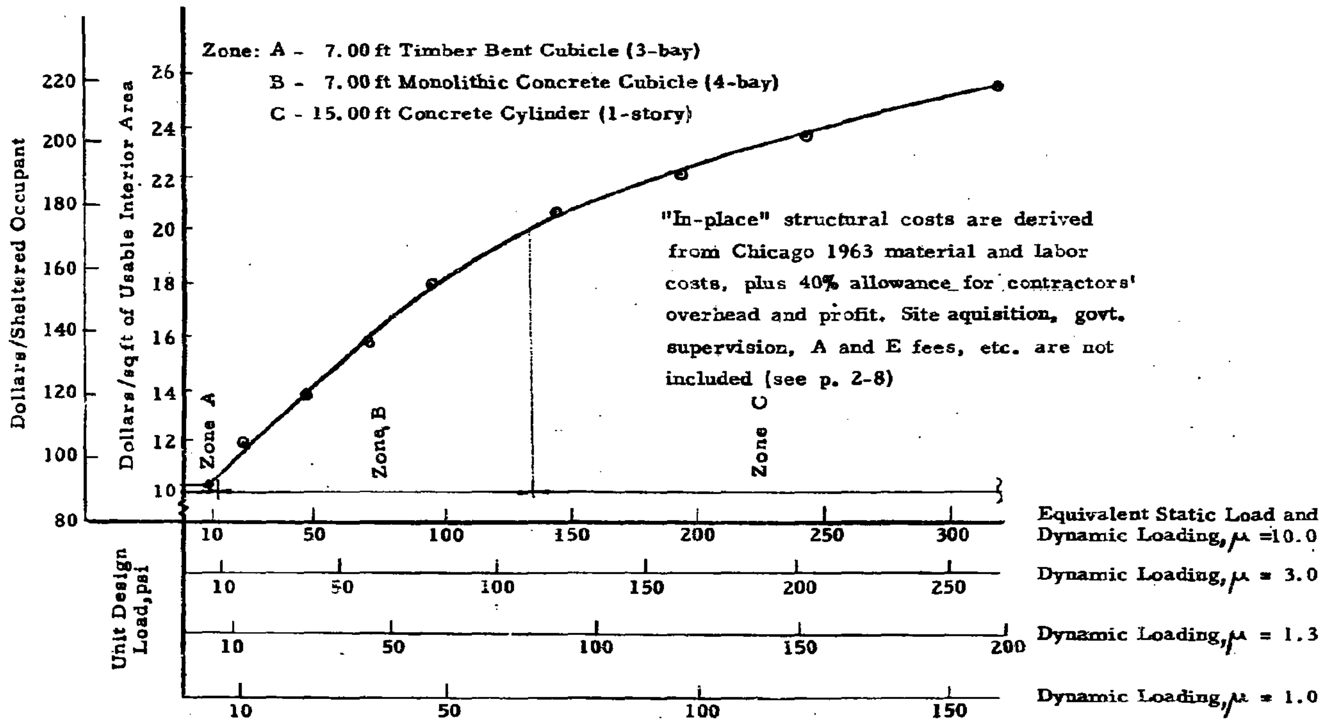

Figure S-1

MINIMUM IN-PLACE STRUCTURAL COSTS
FOR HARDENED 100-MAN CAPACITY SHELTER
(Includes Entrance Ways and Excavation)

Chapter 4 contains several examples of structural shelter design, applying the theoretical relationships supplied in Chapter 3. These Trial Designs are representative of those used to construct the "minimum structural costs" curve of Figure S-1. Finally, Chapter 5 contains a brief summary of the minimum-cost design experience obtained from Chapters 3 and 4, and supplies in-place structural cost data for various configurations of 100-man shelters within the 10 psi to 200 psi range of design overpressure.

Two appendices are attached to the report. The first describes the findings of a limited series of static and dynamic loading tests on buried small-scale roof panels. The second appendix provides a brief "state of the art" discussion of fiber-reinforced plastic shelters.

FINDINGS

The major findings of the study are as follows:

(1) The configurations corresponding to least in-place structural cost for the 100-man shelter consist of a rectangular cubicle for design overpressures less than approximately 100 psi, and of a horizontal cylinder for greater design overpressures within the 200 psi study limit. There is some indication that the optimum configuration at considerably higher design overpressures might be the sphere, but this hypothesis was not verified within the limits of the study.

(2) The structural arrangement corresponding to minimum in-place structural cost is related to the shelter configuration and to the design overpressure. For the cubicle configuration, the use of short spans for flexural members (within the physical limits permitted by interior layout requirements) is generally consistent with maximum economy.

(3) Of those structural materials subjected to detailed examination, the one associated with minimum in-place structural cost for any constant allowable ratio of total strain to elastic strain*

* The ratio of maximum deflection of a dynamically loaded member (a combination of elastic and of plastic strain) to the deflection of the member at its elastic yield point is designated by the symbol μ. For a material with no tolerable range of plastic yielding, $\mu = 1.0$. (See Ref. 2 of report).

is almost invariably reinforced concrete. A limited exception occurs in the lower range of design overpressures (p_{so} = 10 psi) where slight economic advantages result from the use of structural steel, timber, and/or block masonry units. For higher design overpressures, however, an apparent cost penalty is associated with the use of structural materials other than reinforced concrete.

(4) There remains a basic question as to whether the apparent economic advantage of reinforced concrete as a structural material for shelters exists in fact. While this question must remain largely unanswered, due to deficiencies in our current knowledge of blast loading and material behavior, at least two further points should ultimately be considered. First, there is a real question as to the extent of plastic yielding under blast loading which can safely be tolerated by different structural materials and by different shelter configurations. If, for example, it were found that an arch fabricated from steel plate could be designed for μ =10.0 with the same assurance as a geometrically-similar reinforced concrete arch could be designed for μ =2.0, then the relative economics of curved steel plate and of reinforced concrete shells must be re-evaluated. (See Figure S-1, noting relationship between "in-place cost" and assumed value of μ).

Next assuming that a better understanding can be developed as to actual soil-structure interaction during blast loading, the flexibility of a loaded structure within its stability limits may become of major importance in the economic development of a combined soil-structure system to resist an applied load. This possibility is discussed in Appendix A of the report. Should such prove to be the case, consideration must then be given to the flexibilities of structural materials in the relation to each other and to the soil surrounding a shelter.

(5) Lacking a much better understanding of soil-structure interaction and of the economic factors involved in developing a combined soil - structure resistance to applied loading, no quantitative

conclusions can be drawn regarding the economic desirability of soil stabilization in the shelter vicinity.

(6) Fiber-reinforced plastics, while of probable long-range interest for shelter applications, do not appear to be economically-advantageous as a shelter structural material at the present time.

(7) As noted in Chapter 1 of the report, the actual precision of the cost comparisons is limited by the approximations of the simplified loading theories employed in the structural analyses. Within the limits of these approximate theories, it is reasonable to anticipate that further refinements in the structural detailing might lower shelter structural costs by perhaps an additional 5 percent (for example, further attention could be directed to the empirical postulates that ϕ_c (minimum) = 0.25 percent and ϕ_v (minimum) = 0.50 percent in flexural members where web reinforcement is employed). By consideration of more economical construction techniques (pre-casting, multiple use of formwork, more efficient excavation techniques, etc.), the total structural cost for a specific shelter type might conceivably be reduced by another 10 percent. The largest single factor influencing shelter costs for a given capacity and design overpressure is, however, the stated requirements for interior shelter layout. These requirements, which involve the projected behavior of people in s . lters, merit considerably more attention than has been possible in this study.

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NOMENCLATURE

NOMENCLATURE

a	=	portion of half-thickness of masonry wall which, for an angular rotation, θ, is not in contact with the supports, (in.)
A	=	cross-sectional area, (sq in.)
A_s	=	total area of reinforcing steel in a concrete member, (sq in.)
A'_s	=	area of tensile reinforcing steel in a concrete member, (sq in.)
A''_s	=	area of compressive reinforcing steel in a concrete member, (sq in.)
A_{te}	=	area of temperature reinforcing steel in a concrete member, (sq in.)
A_v	=	area of diagonal tension reinforcement steel in a concrete member, (sq in.)
A_w	=	net area of steel beam web, (sq in.)
b	=	width of beam or column, flange width of steel beam, width of one-way slab, (in.)
b'	=	width of stirrup in reinforced concrete member, (in.)
B	=	center-to-center spacing of beams, (ft)
B_T	=	total width of rectangular structure, (ft)
c	=	cohesive strength of soil, (psf)
C	=	general term for cost factor

C_c = cost factor per unit of structural element for concrete, ($/ft or $/sq ft)

C_f = cost factor per unit of structural element for form work, ($/ft or $/sq ft)

C_s = cost factor per unit of structural element for reinforcing steel, ($/ft or $/sq ft)

C'_s = cost factor for reinforcing steel per unit of two-way reinforced slab, between drop panels, ($/sq ft)

C_{st} = cost factor per unit of structural element for temperature steel, ($/ft or $/sq ft)

C_t = factor for composite cost per unit of a structural element, ($/ft or $/sq ft)

C_v = cost factor per unit of structural element for shear reinforcement steel, ($/ft or $/sq ft)

C_w = cost factor for timber per unit of structural element, ($/ft)

C_C = total cost of concrete in structural element, ($)

C_F = total cost of form work for structural element, ($)

C_S = total cost of main reinforcing steel in structural element, ($)

C_{ST} = total cost of temperature steel in structural element, ($)

C_T = composite cost of entire structural element, ($)

C_V = total cost of shear reinforcement steel in structural elements, ($)

C_W = total cost of timber in structural element, ($)

d = effective depth to steel in reinforced concrete members, (in.)

d' = distance from tension face of beam or slab to center of gravity of tension reinforcement, $d' = D - d$, (in.)

d_p = effective depth to steel in drop panel for flat slab design, (in.)

d_w = net depth of web in steel beams, (in.)

D = total depth or thickness of a member, (in.)

D_c = capital diameter, (ft)

D_d = total depth of drop panel in flat slab design, (in.)

e_{cm} = unit strain in arching masonry wall

e'_{cm} = ultimate unit strain in arching masonry wall

e_d = eccentricity of dynamic thrust applied to eccentrically-loaded reinforced concrete column or bearing wall, (in.)

e_{db} = value of e_d at which full resistance of eccentrically-loaded compressive member is simultaneously developed in compression and in tension, (in.)

E = modulus of elasticity, (psi)

f'_c = unit static compressive strength of concrete, based on standard 28-day cylinder test, (psi)

f'_{cm} = unit ultimate strength of arching masonry wall, corresponding to ultimate strain e_{cm}, (psi)

f'_{dc} = unit dynamic compressive strength of concrete, (psi)

f'_{df} = dynamic yield stress in flexure for timber member, (psi)

f'_{dpp} = dynamic yield stress in compression for timber member axially-loaded parallel to the grain, (psi)

f'_{dpr} = dynamic yield stress in compression for timber member axially-loaded perpendicular to the grain, (psi)

f'_{dvh} = dynamic yield stress in horizontal shear for timber member, (psi)

f'_{dvv} = dynamic yield stress in vertical shear for timber member, (psi)

f_{dy} = dynamic yield stress of steel in tension or compression, (psi)

f_f = conventional working stress for static flexural loading of timber, (psi)

f_{pp} = conventional working stress for static compressive loading of timber parallel to the grain, (psi)

f_{pr} = conventional working stress for static compressive loading of timber perpendicular to the grain, (psi)

f_y = static yield stress of steel in tension or compression, (psi)

g = acceleration of gravity, (ft/sec/sec)

h = depth from ground surface to top of structure, (ft)

h_{av} = average depth of earth cover, (ft)

H = height of column, (ft)

I	=	moment of inertia, (lb/in.^{4})
k	=	general term for a defined constant
k_c	=	composite unit cost of concrete in roof slabs and beams, ($)
k'_f	=	composite unit cost of form work in roof slabs and beams, $k'_f = X_f + 0.012\,D$, ($)
k_{sc}	=	diagonal tension coefficient for a slab or beam
k_v	=	shear coefficient for a slab or beam
k_h	=	ratio of horizontal to vertical soil pressure
K	=	general term for a defined constant
L	=	span length of beam, transverse dimension of footing, (ft)
L'	=	length of segment of arch, (ft)
L_{cr}	=	unbraced length of beam on one side of plastic hinge, (ft)
L_{ep}	=	length of steel beam, loaded to incipient yield in shear, at which the maximum elastic moment, M, is numerically equal to the reduced plastic moment, M_{pr}, (ft)
L_{fv}	=	length of beam at which incipient yield in flexure and in shear occur simultaneously, (ft)
L_L	=	length of long span for two-way reinforced concrete slab, (ft)

L_S = length of short span for two-way reinforced concrete slab, (ft)

L'_S = transformed length of short span for orthotropic two-way slab, (ft)

L_T = total length of structure, (ft)

M = general term for applied or resisting moment, (in.-lb)

M_{du} = ultimate dynamic resisting moment of eccentrically-loaded reinforced concrete compression member, (in.-lb)

M_e = elastic resisting moment, (in.-lb)

M_p = fully-plastic resisting moment, (in.-lb)

M'_p = fully-plastic resisting moment with axial load, (in.-lb)

M_{pr} = plastic resisting moment of flanges of a rolled steel beam whose web is fully plastic in shear, (in.-lb)

n = $d/12\,L$, non-dimensional term used in analysis of arching masonry wall

p = ratio of area of steel to net section area, bd, of reinforced concrete member

p_m = peak value of loading pressure on structure, (psi)

p_{so} = peak side-on overpressure at ground surface, (psi)

p_t = ratio of total area of main reinforcing steel to gross section area, bD, of reinforced concrete compressive member

p'_t = ratio of total area of main reinforcing steel to net section area, bd, of reinforced concrete compressive member

P = thrust in arch or column, column load on square footing, wall load per lineal foot on continuous footing, (lb)

P_{db} = ultimate dynamic capacity of eccentrically-loaded reinforced concrete compression member with load eccentrcity e_{db}, (lb)

P_{do} = ultimate dynamic compressive strength of an axially-loaded reinforced concrete compression member, (lb)

P_{du} = ultimate dynamic compressive strength of an eccentrically-loaded reinforced concrete compression member, (lb)

P_{dy} = dynamic yield resistance of an axially-loaded steel or timber compression member, (lb)

P'_{dy} = dynamic yield resistance of an eccentrically-loaded steel compression member, (lb)

P_p = width of square drop panel in flat slab design, (ft)

P_r = perimeter of reinforced concrete column, (ft)

P_y = static yield resistance of axially-loaded steel or timber compression member, (lb)

q = unit yield resistance of member, general, (psi)

q_c = unit compression mode resistance, (psi)

q_d = ratio of pf_{dy}/f'_{dc} for reinforced concrete member

q'_d = ratio of $p f_{dy}/f'_c$ for reinforced concrete member

q_{dt} = ratio of $p_t f_{dy}/f'_{dc}$ for reinforced concrete member

q'_{dt} = ratio of $p'_t f_{dy}/f'_{dc}$ for reinforced concrete member

q_f = unit flexural mode resistance, (psi)

q_{sc} = unit diagonal tension or shear compression mode resistance, (psi)

q_u = unit ultimate resistance, (psi)

q_v = unit shear mode resistance, (psi)

Q = statical moment of the cross-sectional area of a timber beam above or below the neutral axis, (in.^{3})

r_g = radius of gyration, (in.)

r_u = moment arm for internal thrust, P, developed in an arching masonry wall due to an angular rotation θ, (in.)

R = $36\left[\frac{e_c L^2}{d^2}\right]$ Non-dimensional term used in analysis of arching masonry wall. Also roentgen, referring to nuclear radiation.

s = spacing of vertical stirrups for diagonal-tension reinforcement, (in.)

S = elastic section modulus of beam, (in.^{3})

S_L = span of arch or cylinder, diameter of dome or sphere, (ft)

t = thickness of steel plate, (in.)

t_d = effective duration, (seconds)

t_f = thickness of steel beam flange, (in.)

t_r = rise time of pressure pulse, (seconds)

t_w = thickness of steel beam web, (in.)

T = natural period of vibration, (seconds)

u = $\frac{w_o}{d}$ Non-dimensional term used in analysis of arching masonry wall.

v = average shear stress, (psi)

v_{dy} = average dynamic shear yield stress, (psi)

V = total shear, (lb)

V_p = total shear causing full plastification of net web area of rolled steel beam, (lb)

V_u = ultimate shearing resistance of cross-section of reinforced concrete member, (lb)

w = weight per unit of structural element, (lb/ft, lb/sqft or lb/ft^3); also, unit weight of soil, (lb/ft^3)

w_o = maximum deflection of arching masonry wall corresponding to an angular rotation, θ, (in.)

W = weapon yield in megatons

X = general term for cost coefficient

X_c = unit cost of concrete, (\$/ft^3)

X_{cm} = unit cost of reinforced concrete masonry units, ($/sq ft)

X_f = unit cost of form work, ($/sq ft)

X_s = unit cost of steel, excluding shear reinforcement, expressed as ($\$/ft^3$) for reinforcing tie and temperature steel, ($/lb) for rolled steel shapes and ($/sq ft) for steel plate

X_v = unit cost of shear reinforcement steel, ($\$/ft^3$)

X_w = unit cost of timber, ($/MBF)

y = distance measured parallel to y-coordinate axis, (in.)

y = distance to centroid of area, measured parallel to y-coordinate axis, (in.)

z = depth below ground surface, (ft)

Z = plastic section modulus of steel beam, ($in.^3$)

Z_r = reduced plastic section modulus of steel beam, ($in.^3$)

α = ratio of short to long spans of two-way isotropic and orthotropic slabs; also, effective column length

α' = transformed ratio of short to long spans of a two-way orthotropic slab

δ = change in length of an element of an arching masonry wall, (in.)

δ_o = maximum change in length of an element of an arching masonry wall, (in.)

θ = horizontal angle

θ' = ratio of negative to positive reinforcement percentages

μ = ductility factor, coefficient of orthotropy for two-way orthotropic slab, ratio of maximum deflection to deflection at yield.

μ_e = value of coefficient of orthotropy for maximum weight economy in a two-way orthotropic slab

ϕ = general term for percentage of tensile steel reinforcement

ϕ' = general term for percentage of compressive steel reinforcement

ϕ_c = effective percentage of tensile steel reinforcement at mid-span

ϕ_e = effective percentage of tensile steel reinforcement at support

ϕ_{Lc} = effective percentage of tensile steel reinforcement at mid-span in long direction of two-way slab

ϕ_{Le} = effective percentage of tensile steel reinforcement at supports in long direction of two-way slab

ϕ_{Sc} = effective percentage of tensile steel reinforcement at mid-span in short direction of two-way slab

ϕ_{Se} = effective percentage of tensile steel reinforcement at supports in short direction of two-way slab

ϕ_t = total percentage of main reinforcing steel, referred to gross concrete area, bD

ϕ'_t = total percentage of main reinforcing steel, referred to net concrete area, bd

ϕ_{te} = percentage of temperature reinforcing steel and/or tie steel

ϕ_{v} = percentage of web reinforcing steel

STRUCTURAL MATERIALS FOR HARDENED PERSONNEL SHELTERS

CHAPTER 1 INTRODUCTION

1.1 Objectives of the Study

The objective of this research program, as defined by the Office of Civil Defense in its initial request for research proposals, is the evaluation of significant properties, availability, and in-place costs* for those structural materials which might be utilized in a large-volume effort to construct underground group shelters. It was anticipated, in initiating this study, that its execution would indicate possible cost-reduction features in the design of the basic shelter structure. Also, by identifying a broader range of suitable structural materials, national capabilities for a major shelter construction program can subsequently be evaluated.

1.2 Scope

1.2.1 Structural Materials

As a general policy, evaluations are performed for basic construction materials rather than for proprietory units or combinations. Structural materials are subjected to a preliminary screening by applying several qualitative criteria. It is stipulated that any material, in order to qualify for detailed study, must have major physical properties which are suited to the proposed use. It is considered necessary that a material be presently available for construction purposes, without excessive cost, and be of significant commercial importance.

A description of the structural materials which are examined in this study, with pertinent data relative to their significant properties, availability and cost, is contained in Chapter 2.

1.2.2 Shelter Charcteristics

A "hardened" group shelter is designed for use during the attack and early post-attack periods. Its function in providing protection during

*The term "in-place cost," as used in this report, is defined on page 2-8.

attack will normally control its structural design. It must continue to shelter people during the period immediately following an attack, and thus must contain certain basic supplies and equipment. This latter requirement, by establishing minimum space needs for some defined period of occupancy, will usually control the interior layout for the shelter. Other planning considerations, such as alternative peace-time uses, may warrant the provision of facilities or supplies which exceed the minimum requirements. For purposes of this study, however, attention is given only to such shelter features as are considered essential for human survival. Further, by reason of the defined research objectives, interest is restricted solely to those uses and functions which may influence the selection of structural materials for the shelter.

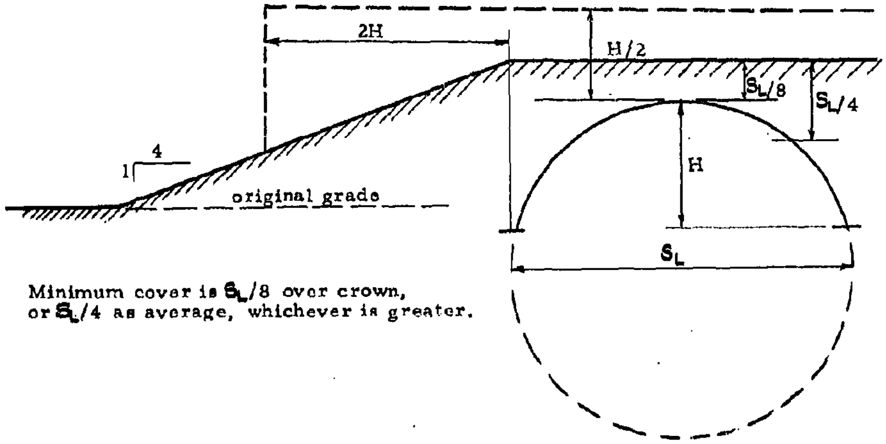
The research program postulates group shelters which satisfy the requirements for "fully-buried" structures,[1,2]* as indicated in Figure 1-1. These shelters are to be located at depths below normal ground surface which, at their maximum, do not exceed 50 feet to the foundation level. Thus, the shelters may also be considered as "shallow-buried" structures[3]. The analyses further assume that the structures will be located above the level of the ground water.

Associated with the "shallow" burial of the shelters and the stipulation that all portions of the structure be located above the ground water level, it is assumed that open-cut excavation methods will be employed during construction. The shelters will then be assembled or fabricated in-place, and the excavation backfilled under controlled conditions.

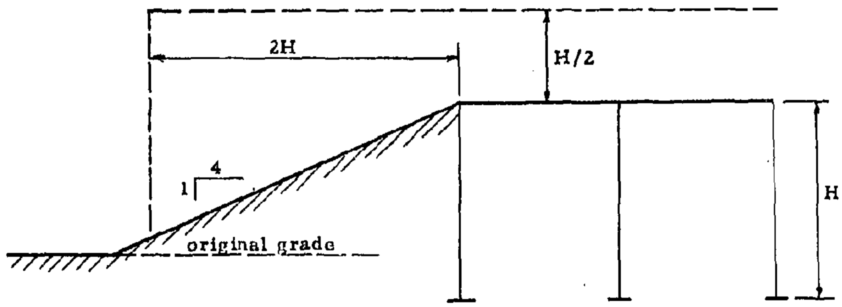
1.2.3 Attack Environment

In order to evaluate the structural materials for use in underground group shelters, it is first necessary to identify the characteristics of those nuclear weapons which might be used in an attack. Thus, finite levels of thermonuclear explosions of fixed magnitudes are assumed at the onset

*Superscript numerals in parentheses refer to references listed in the bibliography included as pages 6-1 to 6-4 of this report.



(a) ARCH, DOME, CYLINDER OR SPHERE



(b) RECTANGULAR STRUCTURE

Boundaries of minimum cover.

Boundary of region within which the maximum recommended slope is 1 on 2. The additional cover of this region may be required for radiation protection or may be expedient because of terrain conditions.

Figure 1-1

MINIMUM EARTH COVER REQUIRED TO DEFINE A STRUCTURE AS FULLY-BURIED

of the study. The types and magnitudes of destructive effects [4] resulting from these explosions are then identified. Finally, the behavior of structural elements and materials in an environment containing these weapons effects is analyzed.

The structural design of a buried shelter, and hence the range of usefulness of a given structural material, is largely controlled by dynamic explosion effects. These consist of direct ground shock close-in to the explosion center, and ground shock initiated by air overpressure at the ground surface. Excluding direct ground shock from further consideration, on the assumption that a system of shock isolation will be provided where this effect becomes critical, the dynamic load on a buried structure is then related to the force-time load variations induced by surface overpressures. Simplified theory [1, 3, 5] can be used to relate the magnitude and duration of surface overpressure, the type and depth of soil above a buried structure, and the effective loading on a structural element whose elasto-plastic characteristics are specified. The duration of the effective loading from megaton yields is sufficiently long, in comparison with the natural period of vibration for most structural elements, that the size of weapon need no longer be treated as a separate variable. By introducing this simplification, a generalized analysis becomes feasible since the design loading for a structural element can be related to the level of overpressure without specifying the weapon yields. Finally, for the depths of burial associated with shallow-buried structures, any attenuation of peak overpressures due to damping within the cover soil is considered to be negligible. The peak dynamic pressure at the ground surface can thus be taken as a reasonable approximation of the actual loading on buried structural elements due to the explosion of megaton weapons. This dynamic loading, with suitable modifications based on the elasto-plastic characteristics of the structural material and on its permissible yield deflections, can then be used in estimating the resistance which a structural member must provide. If subsequently desired, such preliminary designs can be refined by introducing considerations of direct ground shock, rise and duration times for the pressure wave induced in the ground by the air blast, and a broad category of inadequately-understood effects described as "pressure wave-soil-structure interactions".

The simplified theory of blast wave loading and structural response, as described, is applied in this study of structural materials. Megaton weapons are assumed, with yields from one to 100 MT and surface side-on overpressures of 10 psi to 200 psi. Structural elements, insofar as their component material properties permit, are designed to resist loading within these ranges. As an intermediate step, these elements are initially designed for uniformly-distributed static loadings. Conversion factors are then indicated which, within the assumptions of this simplified loading theory, may be used to equate the anticipated dynamic loading to an equivalent static resistance. If so desired, any alternative theory[6, 7] relating dynamic loading to equivalent static resistance could be applied with equal simplicity.

1.3 Methodology

The research program is formulated to evaluate the suitability of selected structural materials for use in underground group shelters. The investigative techniques which are applied in meeting this objective are described briefly in this section. The detailed computations resulting from their application are supplied in subsequent chapters.

After some initial study, it was concluded that material performance could most usefully be evaluated in terms of "in-place" costs. Certain preliminary qualifications become necessary, however, since there is no unique relationship between the in-place cost of a structural material and the contribution which it actually provides to total structural resistance. Structural materials, by virtue of inherent anisotropic properties, normally cannot supply equal resistance in all directions to applied stress. Analyses of structural materials and structural elements can, of course, be formulated on the assumption that maximum utilization will be made of inherent material strength properties. Such analyses are primarily of academic interest, unless the resultant structural forms and required conditions of loading can be reconciled with actual conditions in a practicable structure. Furthermore, a design for maximum strength utilization will not in itself ensure minimum cost, since incremental costs of fabricating an optimum-strength component may exceed incremental savings in material costs.

When actual design possibilities are analyzed, involved interrelationships are generally found between the in-place cost for a particular structural material and such diverse factors as exterior and interior architectural layouts, details of the structural system, and the magnitudes and distribution of applied loads. As a consequence, unless initial steps are taken to evaluate the major factors influencing structural costs, direct comparisons of in-place costs for structural materials will have little practical significance. Analyses in this latter category may provide a useful cost comparison of over-all structural designs, but are of limited value in evaluating the absolute merits of the constituent materials.

These general concepts are applied in this evaluation of structural materials. Initial assumptions are made as to the range and magnitude of weapons effects, loading distribution on the buried structures and controlling dimensions for interior shelter layout. Subsequently, subject to any restrictions introduced by these assumptions, each structural material is analyzed for a range of possible uses.

Additional study limits are established by restricting the investigation to three basic shelter configurations. These consist of the rectangular single-story cubicle, the horizontal cylinder or associated semi-circular arch, and the sphere or associated hemispherical dome. These three configurations have been the subjects of past analytical studies and of limited field testing[4, 8]. As such, they constitute a valid sampling of current concepts in shelter design. The combinations of plane and curved surfaces introduced by these configurations facilitate the rapid comparison of a variety of structural systems and elements. By comparing the costs of the several design alternatives, the most favorable "in-place" cost is approximated for each structural material.

In summary format, the investigative techniques for this study involve the following steps:

1) The program is planned to obtain a comparison of in-place structural costs for "fully buried" group shelters, constructed from selected structural materials. The shelters so considered are assumed to be located at "shallow" depths and above the

elevation of ground water. Weapons yields of one MT to 100 MT, producing side-on surface overpressures of 10 psi to 200 psi at the shelter locations, are postulated as attack environments for analyses of structural elements.

2) The shelter designs considered in the study are required to satisfy certain criteria pertaining to attack environment and conditions of use. General standards adopted for interior layouts are described in Chapter 4.

3) Basic shelter configurations are selected to provide reference frames for the structural material evaluations. These configurations consist of the one-story rectangular cubicle, the horizontal cylinder or semi-circular arch, and the sphere or hemispherical dome.

4) The explosion effects predicted for thermonuclear yields of one MT to 100 MT have been examined for their probable influence on the structural design of a buried shelter. It is concluded that the minimum depth of cover over a shallow-buried structure may be controlled either by the requirement for radiation protection or by the criteria established for "full burial". Once the controlling cover requirements are satisfied, however, the structural design of any specified buried shelter can be closely related to the air-blast induced ground pressure at the structure level. Finally, by utilizing conventionalized approximations of actual loading conditions, the surface overpressure is related to an "equivalent" uniform static pressure on a buried structural element. Structural loading is thus considered to be a direct function of overpressure, irrespective of the yield of weapon.

5) Assumptions are made as to the distribution of loading on each buried shelter configuration, again following the simplified loading theories proposed by other writers [1, 3, 5]. The effective loads on a fully-buried cubicle are thus considered to act normal to its plane surfaces. The full surface overpressure on a buried cubicle is assumed to act normal to its horizontal surfaces, while the normal loading on its

vertical side and end walls is taken as some fraction of the vertical loading. The actual relation between the horizontal and vertical loading, for a given shelter, is presumably dependent upon soil properties and the location of ground water with respect to the structure[3]. Singly and doubly curved surfaces, such as occur in the arch and the dome, are assumed to be axially-loaded in compression at incipient failure. This postulates that the soil adjacent to such curved surface will restrain the various possible buckling modes until the ultimate compressive strength of the member has been fully developed. The possibility of minor localized flexural stresses is recognized, to a limited extent, by specifying a minimum value for the flexural reinforcement in concrete compressive members.

6) Feasible structural systems, compatible with the prior assumptions as to type and pattern of loading on a buried shelter, are identified at the onset of the study. Possible structural systems are classified as unframed, partially framed, or fully framed. Structural elements associated with each framing system are identified and described. Tables 1-1 and 1-2 provide summaries of the structural systems and elements for the three basic shelter configurations.

7) A preliminary selection of suitable materials is made by applying the generalized criteria described in Section 1.2.1. Static and dynamic strength properties are identified and recorded for each structural material. As an aid in this selection, letters were prepared and mailed to 28 representative material producers and trade organizations, requesting any relevant information pertaining to the use of each material in a blast-resistant buried shelter. Fifteen replies were received, nine of which were accompanied by some type of design data.

8) Analyses are performed for each suitable use of a structural material in a structural element. Prior assumptions are used to identify the function of the element in the structural

TABLE 1-1 CLASSIFICATION OF STRUCTURAL SYSTEMS FOR SELECTED SHELTER CONFIGURATIONS

Configuration of Shelter	Description of Structural System: No Framing	Partially Framed	Fully Framed
Rectangular or Cubicle	No separate framing system. Roof (and floor, if integral) resists axial thrust in two coordinate planes due to bearing reactions of laterally-loaded walls, plus transverse blast loading in third plane, as roof spans between bearing walls. Walls resist axial thrust in one plane due to bearing reaction of roof (and floor, if integral), plus axial thrust in second plane due to bearing reaction of adjacent walls, plus transverse blast loading. Foundations can be integral slab or wall footings, with provision to transfer lateral thrusts.	Roof (and floor, if integral) spans between rectangular bents. Walls normally span between roof and floor: thus, roof resists axial thrust in one (or two) planes due to bearing reaction of laterally loaded walls, plus transverse blast loading in third plane as roof spans between bents. Bents resist axial thrust from side walls, plus transverse loading due to reaction of loaded roof. Foundations are as described for unframed structure, plus column footings.	Full framing system of columns, beams and girders. Columns resist axial thrust from roof and transverse loads from walls. Beams and girders resist transverse loads from roof plus axial thrust from walls. Walls and roof (and floor, if integral) resist transverse loads only. Foundations are as described for partially-framed structure.
180° Barrel Arch or Full Cylinder	No separate framing system. Singly-curved shell, with assumption of uniform radial loading in plane of shell curvature, resists axial thrusts in plane of arch plus axial thrust in second plane due to end walls. End walls span between structure perimeter and floor. Arch foundation can be integral floor (which will transmit moments to shell) or continuous footings. No foundation required for cylinder.	Arch or cylinder shell spans between circular or segmented bents. Shell carries axial thrust due to assumed radial loading, probably with localized flexural and shearing stresses adjacent to stiffener ribs. Ribs support axial thrust, probably supplemented by transverse loading from the shell. Localized moments in segmented bent due to eccentricity of thrust. Foundations are as for unframed structure, plus column footing.	Longitudinal beams span between stiffener ribs. Ribs resist axial thrust, plus transverse loading from shell. Beams resist transverse loading from shell plus axial thrust from end walls. Shell acts primarily in flexure, spanning between ribs and beams. Foundations are as described for partially-framed structures.

TABLE 1-1 CLASSIFICATION OF STRUCTURAL SYSTEMS FOR SELECTED SHELTER CONFIGURATIONS (Cont'd)

Configuration of Shelter	Description of Structural System		
	No Framing	Partially Framed	Fully Framed
180° Dome or Full Sphere	No separate framing system. Doubly-curved shell, with assumption of uniform radial loading in both planes of shell curvature, resists axial thrust in two planes. Dome foundation can be integral floor (which will transmit moment to shell) or continuous footing. No foundation required for full sphere.	No distinction between partial framing and full framing for this configuration. Curved ribs resist axial thrust plus transverse loading from shell. Curved beams between ribs (if provided) will resist similar forces. Shell resists axial thrust and probably localized flexural and shear stresses adjacent to ribs and beams. Foundations are as described for unframed structure, plus column footings.	

TABLE 1.2 STRUCTURAL ELEMENTS FOR BURIED SHELTERS

Structural Element	Primary Structural Material	Function in Structure
Column (axially-loaded or eccentrically-loaded)	Rolled steel section, reinforced concrete, timber post	Primarily to resist vertical loads, possibly lateral loads in addition. Used in partially-framed or fully framed structure.
Beam or beam column	Rolled steel section, reinforced concrete, timber beam	Primarily to resist transverse loads, possibly axial thrust in addition. Used in partially-framed or fully-framed structure.
Axially-loaded bearing wall.	Reinforced concrete, reinforced concrete masonry units.	Primarily to resist axial loads, frequently lateral loads in addition. Used in all structural systems.
Eccentrically-loaded bearing wall	Reinforced concrete	Same as above, except that thrust is no longer axial.
One-way slab, two-way slab	Reinforced concrete	Primarily to resist transverse loads, possibly axial thrust in one or two planes. Used for roof, floor, possibly end walls.
Flat slab	Reinforced concrete	Primarily to resist transverse loads. Used with columns in a partially-framed structure.
Singly-curved compression member	Reinforced concrete, corrugated steel plate, uniform-thickness steel plate, fiber-reinforced plastic	Primarily to resist axial thrust. Used as shell for arches and cylinders.
Doubly-curved compression member	Reinforced concrete, uniform-thickness steel plate, fiber-reinforced plastic	Primarily to resist axial thrust. Used as shell for dome or sphere.

TABLE 1-2 STRUCTURAL ELEMENTS FOR BURIED SHELTERS (Cont'd)

Structural Element	Primary Structural Material	Function in Structure
Filler panel	Reinforced concrete, reinforced concrete masonry units, timber sheathing	Primarily to resist lateral loading. Spans between framing members of a fully-framed structure
Footings (isolated and continuous)	Reinforced concrete	Transmit wall or column loads to soils. Used for all structural systems.
Isolated floor slab	Reinforced concrete	Supports conventional floor loading. Used when exterior loads are carried by footings.

system, the pattern of loading on the structure, and the magnitude of load from each postulated weapon yield. Controlling failure modes are identified for each use and each material, and generalized strength equations are formulated.

9) Estimates are made of the in-place cost for each structural element. These costs are then analyzed for each assumed condition of use, to identify the major parameters which influence structural cost. In specific examples, generalized cost equations are formulated and minimized to establish least-cost design relationships. In other cases, repetitive trial solutions are used to approximate these least-cost relationships.

10) Weapons effects, other than overpressure, which are associated with one MT to 100 MT thermonuclear weapons and levels of side-on surface overpressure in the 100 psi to 200 psi range, have also been studied. The most important of these remaining effects, from the viewpoint of structural design, is the total nuclear radiation associated with each weapon yield. Accordingly, a maximum effective dose of 50 R and a two-week shelter stay are postulated for design purposes. Approximate relationships, giving consideration to time-decay of radiation intensity and to distance from the radiation source, are used to obtain estimates of initial and fallout radiation for various weapon yields [4]. The shelter surface is then approximated as a plane shield exposed to radiation energy, and its required mass calculated for each size of weapons. This mass requirement, for fixed levels of overpressure, is expressed as an equivalent depth of earth cover (see Chapter 4). The equivalent earth depth can, with reasonable accuracy for this study, be related solely to overpressure level.

11) Alternative interior layouts for each basic configuration of shelter are then examined. Each layout provides space for 100 men and meets specific criteria as to minimum interior dimensions, required bunk and aisle areas, and requirements for operational space. A few preliminary calculations usually

serve to indicate the relationships between interior layout and structural costs which exist for given shelter configurations and structural systems. By this procedure, the interior layout associated with minimum structural cost can be identified for each practicable combination of structural material, shelter configuration and attack environment. The selection of layouts is guided, to a considerable extent, by the findings of recent investigations of optimum shelter design [9, 10].

12) The in-place structural costs are estimated for entire shelters. The shelters (whose designs are based on the minimum-cost studies) are synthesized from the structural elements. By an iterative process, considering a range of equivalent static loads, trends in relative shelter costs are identified and minimum structural costs are determined.

13) The cost studies identify those combinations of structural materials, framing system and shelter configuration which will result in minimum structural cost at each level of loading. Structural materials of significant importance in shelter construction are thus identified. A review of the historical availability of these materials is included, with limited projections as to their probably future availability.

1.4 Limitations of the Study

The objective of this study is an evaluation of structural materials in group shelters. This evaluation is performed by postulating conditions of use for each material, and comparing materials on the basis of their in-place costs. The investigative techniques require the preparation of preliminary designs and cost estimates. This estimating process, for any type of proposed construction, will necessarily involve a degree of uncertainty. However, the limitations which are discussed in this section are not related to statistical variations in material properties nor to the inherent variations of competitive bidding. Neither, without attempting to minimize the possible importance of such uncertainties, do these limitations refer to variations between actual weapon parameters and those assumed for analysis. Rather, the inherent limitations of the study are considered to include the following:

1) The estimated shelter costs, and consequently any conclusions drawn therefrom, pertain only to the structural portions of buried group shelters. Items such as blast closures and fittings, mechanical ventilation equipment, communications and monitoring devices, etc., are not included in the cost studies. Their cost, which will constitute a significant portion of total shelter costs, may be strongly influenced by shelter layout, shelter configuration or structural material properties. If such is actually the case, any conclusions based only on structural costs will prove misleading if directly extrapolated to total shelter costs. This suggests the need for evaluations of total shelter costs, perhaps by extending the techniques used herein.

2) Current understanding of blast wave soil-structure interaction is in a rudimentary stage. The simplified theory used in this study assumes that the surface overpressure initiates a ground wave which advances, essentially without attenuation due to its passage through the soil, until it uniformly engulfs a buried structure. The assumed loading on the structure is, to a limited extent, related to the elastic and plastic yield characteristics of the structural materials. In essence, however, it is assumed that the structure responds to a uniform dynamic load which is equal to the peak side-on surface overpressure. More complex analyses recognize the transient nature of the structural loading, but their application generally results in structural designs which are very similar to those obtained by the simplified methods.

In actuality, however, the load reaching a buried structure must be transmitted through the surrounding soil. If failure of the buried structure requires the continued application of load through some finite displacement, the structure and the surrounding soil must act in combination to sustain loads. The extent to which their combined strength will exceed the strength of the structure alone will be dependent upon their

relative stress-strain properties and upon their absolute strengths.

The analyses used in this study follow conventional procedures by assuming that the buried structure must be designed to resist surface loading, without significant strength contribution from the soil. It is thus structural strength, as distinct from soil-structure strength, which is recognized in the evaluation of material costs and of shelter designs. A better understanding of the problem, however, might indicate that the important feature is potential soil-structure strength, rather than structural strength alone (See Appendix A to this report).

To illustrate this point, assume that a shelter is to be designed as a cylinder, fully-buried, with its longitudinal axis placed horizontally. Conventional analyses will assume that the structure supports a uniform radial load prior to failure. The material in the structure will accordingly be designed to resist direct compression, probably with some minor provisions for localized flexural stresses. Structural steel and reinforced concrete, which are typical of the suitable structural materials for this application, will then be compared on a cost basis. The difference between the in-place cost for these materials will be such that, even after design recognition has been given to the ability of the steel to tolerate large strains, the reinforced concrete structure will probably have the lower in-place cost. If it were possible to include the soil contribution to total resistance in the evaluation, however, it is entirely possible that a different conclusion might be reached. The inherent flexibility of the steel structure would favor its use in a soil which could mobilize a large measure of strength at high strains. Conversely, if the soil were such that its shearing strength became less at large strains, the rigidity of the concrete structure would prove advantageous. The inability to identify the possible contributions of soil

strength must, by inference, almost certainly result in misleading evaluations of required structural resistances and of actual loads in blast-loaded buried structures. While this lack of knowledge will generally encourage the over-design of such structures, the margin of design safety which is thus provided will not be consistent. Rather, as implied by the preceeding discussion, it may be strongly dependent upon the soil properties and the soil interaction with the buried structure.

3) Minimum-cost designs are prepared for a variety of structural elements and structural materials. These elements are combined to form shelters of specified layouts and configurations. Estimates are made of the total structural cost for each shelter. Since the initial choice of shelter configurations is admittedly arbitrary, there remains the possibility that some optimum combinations of structural materials and shelter configurations are not considered in the study. Conceivably, there could be other configurations where, within some finite range of design conditions, the structural materials considered in this study could be utilized more economically. There could also be other structural materials which, either for the configurations considered herein or for some entirely different configuration, would be economically preferable. It seems unlikely, however, that the conclusions reached in this study would be significantly altered.

4) The structural elements are designed for minimum cost and subsequently combined in a variety of ways to form 100-man capacity shelters. Interior layouts which satisfy specific operational criteria at least structural cost are used in the cost analyses of each shelter configuration. The structural costs for each shelter, when expressed in terms of design occupants, are strongly dependent upon the interior layout. Thus, the validity of the criteria used in developing these layouts becomes a matter of concern.

5) There is a distinct possibility that the 100-man capacity shelter selected for detailed study is not the optimum size for maximum structural economy. It should not be assumed that the cost relationships which are developed for this size of buried structure must necessarily remain valid if other sizes of shelters are considered. Expressed another way, minimum structural costs are analyzed in this study for specific shelter configurations which are designed to accomodate a fixed number of occupants. Minimum structural costs are not analyzed for these same configurations for cases where the design occupancy is itself treated as a variable. Such studies would obviously be desirable and, following the procedures described in Chapter 4, could readily be performed.

6) The generality of this study is, by initial definition, restricted to fully-buried shelters which are located entirely above the ground water table. It is recognized that these restrictions, in certain areas of the country, will severely limit the applicability of the study findings. An obvious problem arises in areas where the permanent ground water table is at or near the ground surface. Possible alternatives to full burial in such a situation may include relocation to more favorable topography or use of "partially-buried" or above-ground construction. Frequently, the first of these alternatives is logistically impracticable. Site limitations in congested urban areas may also make it impracticable to obtain sufficient space to use "partially-buried" construction. For such cases, if the design overpressure is of a magnitude such that earth protection for the structure becomes essential, the shelter must be placed below ground and designed to remain water-tight at all times. Such a requirement, by introducing major design complexities, will result in increased construction costs.

7) The structural materials are evaluated by a procedure which relates their in-place cost to their ability to withstand loading. Implicit recognition has also been given to their effectiveness

in attenuating nuclear radiations. This latter effect has little influence on costs, however, particularly at the higher over-pressure levels.

There is a remote possibility, not considered in this study, that thermal considerations could influence the selection of structural materials for use in a buried shelter. Two possible situations are visualized. First, if a fire storm were to occur above a buried shelter, heat transmission through the cover soil might cause critical damage to the shelter structures. This possibility is, for many reasons, considered to be improbable. Next, several studies have suggested that temperature rise within the shelter, due to heat released from its occupants and equipment, will constitute a controlling design condition.[2, 9] Heat is removed from the shelter by circulation of air and by conduction through the shelter surfaces to the surrounding soil. It is possible that the substitution of structural materials with improved thermal transmission properties, even if accomplished at increased structural costs, might on occasion be justified by a reduction in the cost of mechanical cooling equipment. Such a possibility is not considered in this study.

1.5 Possible Uses for Study Findings

Two general categories of use are visualized for the findings reported herein. The relationships between structural cost and level of protection will be of immediate use in the broad aspects of passive defense and of shelter planning. The structural cost of providing a quantity of 100-man capacity shelters which meet some specified level of "hardness" can, by introducing regional cost adjustments, be readily determined from the tables and graphs contained in this report. The additional structural cost which would result from an increase in this specified hardness level can be determined in similar fashion. The desirability of extending these studies to include all shelter components is apparent, since planning projections could then be based on total shelter cost rather than on structural cost alone. Also, consideration of shelter sizes other than 100-man capacity considered herein would permit a greater measure of flexibility in planning projections.

The relationships between cost, configuration and design loading, as developed herein, will be of value when selecting the optimum shelter structure for a particular location and function. As detailed design proceeds, reference may be made to the analytical equations which are supplied for the major structural elements. In several cases, where the manual solution of an analytical equation would become time-consuming, repetitive solutions are obtained and tabulated by computer methods. The analytical equations are accompanied by cost equations or expressions which, when translated into current local costs, will serve as an excellent guide to the proper combination of materials for minimum structural costs.

CHAPTER 2 STRUCTURAL MATERIALS

2.1 Materials And Their Properties

2.11 Structural Steel

Structural steel is analyzed as an elasto-plastic material which obeys Hooke's Law up to some defined yield point, and subsequently deforms at constant stress through a plastic range. The stress value corresponding to its yield point will, for static conditions of loading, be dependent upon the material constituents of the steel and upon the method by which the steel is processed and fabricated into structural shapes. Investigations have shown[11, 12] that the yield stress for structural steel becomes greater at increased rates of load application, although the modulus of elasticity, E, remains essentially constant. Based on these earlier findings, it is proposed that dynamic yield stresses for low carbon steels in flexure and in direct tension or compression be taken as 1.25 times the comparable yield stresses for static loading. The dynamic shear yield stress will be taken as 0.6 times the dynamic tension yield value.

The ability of steel to deform plastically, without any appreciable variation in yield resistance during its plastic range, becomes of importance in the design of blast-resistant structures. The use of steels with limited ductility, such as may be occasioned by heat treating or by a high carbon content, should thus be viewed with caution[5]. Particular care is required at welded connections and at stress concentrations adjacent to bolt and rivet holes if brittle fractures are to be avoided. Tables 2-1 and 2-2 list representative structural steels and proposed values for their yield stresses when loaded at rates commensurate with blast loading.

2.12 Steel Reinforcing Rod

The requirements for structural steels in blast-loading applications are, in general, equally applicable to reinforcing steels. Table 2-3 lists standard types of reinforcing bars and their corresponding static yield stresses, as set forth in current ASTM standards[13]. An increase in yield-

Table 2-1

REPRESENTATIVE STRUCTURAL STEELS

PROPRIETARY NAME OF STEEL	TYPICAL SUPPLIER	SPECIFIED STATIC YIELD STRESS, psi	ASTM DESIGNATION	REMARKS
Structural Grade	Bethlehem Steel Co.	36,000	A- 36	Replaces A-7
Structural Grade	U.S. Steel Corp.	36,000	A- 36	Replaces A-7
Structural Grade	Inland Steel Co.	36,000	A- 36	Replaces A-7
Structural Grade	Northwestern Steel and Wire Co.	36,000	A- 36	Replaces A-7
Structural Grade	Phoenix Steel Corp.	36,000	A- 36	Replaces A-7
High Strength "A"	Armco, Sheffield Div.	50,000	A-242	
High Strength "A"	Armco, Sheffield Div.	45,000	A-441	
Manganese Vanadium	Bethlehem Steel Co.	50,000	A-441	
Man-ten (A-440)	U.S. Steel	50,000	A-440	
Shef-ten	Armco, Sheffield Div.	50,000	A-440	
YSW A-441	Youngstown Sheet	50,000	A-441	
Republic A-441	Republic Steel	50,000	A-441	
Jalloy S-100	Jones and Laughlin	100,000	---	Heat Treated
Jalloy S-110	Jones and Laughlin	110,000	---	Heat Treated
N-A-XTRA 100	Great Lakes Steel	100,000	---	Heat Treated
N-A-XTRA 110	Great Lakes Steel	100,000	---	Heat Treated
SSS 100	Armco, Sheffield Div.	100,000	---	Heat Treated
SSS 100A	Armco, Sheffield Div.	100,000	---	Heat Treated
T-1	U.S. Steel, Lukens Steel	100,000	---	Heat Treated
T-1 Type A	U.S. Steel, Lukens Steel	100,000	---	Heat Treated

Table 2-2

GROUPING OF TYPICAL STRUCTURAL STEELS
ACCORDING TO DYNAMIC YIELD STRENGTH

STRUCTURAL CATEGORY	MINIMUM STATIC YIELD STRENGTH psi	ASSUMED DYNAMIC YIELD STRENGTH psi	REMARKS
Heavy Shapes	33,000 - 36,000	44,000	ASTM A-7, A-36
Heavy Shapes	42,000 - 50,000	52,000	ASTM A-242, A-440*, A-441
Heavy Shapes	60,000	60,000	
Plates	33,000 - 36,000	44,000	ASTM A-7, A-36
Plates	50,000	60,000	ASTM A-242, A-440, A-441
Plates	100,000	100,000	Heat Treated

*ASTM A-440 is not recommended for welding.

Table 2-3

KINDS AND GRADES OF REINFORCING BARS AS SPECIFIED IN ASTM STANDARDS

Type of Steel and ASTM Specification No.	Size Nos. Inclusive	Grade Designation	Static Yield Point Min., psi	Elongation in 8", Min. Percent (1)	Cold Bend Test (2)
Billet Steel A-15	2 to 11	Structural	33,000	$\frac{1,200,000}{\text{Tens.Str.}}$ Min. 16%	Under Size No. 6 - 180°, d = 2t Nos. 6, 7, 8 - 180°, d = 3t Nos. 9, 10, 11 - 180°, d = 4t
		Intermediate	40,000	$\frac{1,100,000}{\text{Tens.Str.}}$ Min. 12%	Under Size No. 6 - 90°, d = 3t Nos. 6, 7, 8 - 90°, d = 4t Nos. 9, 10, 11 - 90°, d = 5t
		Hard	50,000	$\frac{1,000,000}{\text{Tens.Str.}}$	Under Size No. 6 - 90°, d = 4t Nos. 6, 7, 8 - 90°, d = 5t Nos. 9, 10, 11 - 90°, d = 6t
Billet Steel A-408	14S, 18S	Structural		13	None
		Intermediate		10	None
		Hard		7	None
Billet Steel 60,000 psi Yield Point A-432	3 to 11	(3)	60,000	$\frac{1,000,000}{\text{Tens.Str.}}$	Under Size No. 6 - 90°, d = 4t Nos. 6, 7, 8 - 90°, d = 5t Nos. 9, 10, 11 - 90°, d = 6t
	14S, 18S	(3)	60,000	7	Nos. 14S, 18S - None
High Strength Billet Steel A-431	3 to 11 14S, 18S	(3)	75,000	Varies with bar size, 5% to 7 1/2%	Size Nos. 3, 4, 5 - 90°, d = 4t Nos. 6, 7 - 90°, d = 5t Nos. 8, 9 - 90°, d = 6t Nos. 10, 11 - 90°, d = 8t Nos. 14S, 18S - None
Rail Steel A-16	2 to 11	Regular	50,000	$\frac{1,000,000}{\text{Tens.Str.}}$ Min. 5%	None
	3 to 11	Special	60,000	$\frac{1,000,000}{\text{Tens.Str.}}$ Min. 5%	None
Axle Steel A-160	2 to 11	Structural	33,000	$\frac{1,200,000}{\text{Tens.Str.}}$ Min. 16%	Under Size No. 6 - 180°, d = 2t No. 6 and over - 180°, d = 4t
		Intermediate	40,000	$\frac{1,100,000}{\text{Tens.Str.}}$ Min. 12%	Under Size No. 6 - 180°, d = 6t No. 6 and over - 90°, d = 6t
		Hard	50,000	$\frac{1,000,000}{\text{Tens.Str.}}$	Under Size No. 6 - 90°, d = 6t No. 6 and over - 90°, d = 6t

(1) For base sizes of deformed bars. See specifications for adjustment for small and large sizes and for values for plain bars.
(2) d diameter of pin around which specimen is to be bent, and t nominal diameter of specimen. Values shown are for deformed bars. See specifications for values for plain bars.
(3) Designated by specification title and number.

point stresses over static values can be anticipated, particularly for low-carbon steels, as a consequence of the rapid application of load.

Four levels of dynamic yield strengths, - 44,000, 52,000, 60,000 and 75,000 psi, - are considered in this study. The 44,000 and 52,000 dynamic yield strength steels correspond to static yield strengths of 40,000 and 50,000 psi respectively. No dynamic yield strength increase is assumed for the 60,000 psi (ASTM 432) or 75,000 psi (ASTM 431) steels since the ductile properties of these steels may restrict their full use. The full continuity of all types of reinforcing steel should be ensured by adequate lapping and by welding. Shear reinforcement for flexural members, where required, should be placed normal to the bending axis. Members with both top and bottom steel, adequately tied, will have greater ductility than singly-reinforced members with the same quantity of positive reinforcement, hence are favored for blast-resistant design. Such, members are described as "doubly reinforced," although the quantities of reinforcement steel in top and bottom need not be the same.

2.13 Structural Concrete

Tests [14] have shown that the ultimate strength of structural concrete becomes greater at increased rates of loading. Thus, for analyses of buried shelters exposed to blast loading, the dynamic compressive strength of the concrete is taken as 1.25 times its comparable static strength. This strength increase is assumed applicable to axially and eccentrically-loaded compression members. However, lacking definitive test data, the dynamic strengths of structural concrete in its shearing modes are equated to the comparable static values. Thus, as indicated in the design equations of Chapter 3, the design of a concrete flexural member is based on the static ultimate strength of the concrete. The unit bond strength of deformed bars in reinforced concrete, under conditions of dynamic loading, is taken as 0.15 f'_c as proposed in earlier studies.[2]

A range of concrete static compressive strengths from 2000 psi to 6000 psi is examined in this study, thus permitting an evaluation of the influence of concrete strengths on estimated in-place costs.

2.14 Masonry Units

The suitability of masonry units for the application considered

in this study is primarily related to the compressive strength of the material. Handbooks and test reports supply widely different values for the compressive strength of masonry. The size of the specimen tested, the type of workmanship, the mortar, and the masonry strength itself are found to be of major importance. It is frequently observed that the crushing strength of masonry material will range between 500 psi and 4000 psi.

In general, the modulus of elasticity for the masonry increases almost linearly with the compressive strength up to some limiting value, beyond which the modulus of elasticity may remain almost constant.

The selection of a proper value of e'_{cm} for a masonry material, where e'_{cm} is the strain associated with the crushing strength f'_{cm}, requires a knowledge of the stress-strain relationships for the material considered. This can best be obtained from a compressive loading test on the material.

Reinforced concrete masonry units of standard 4", 6", 8" and 12" sizes will be studied in some detail. These units will be connected with steel reinforcing rod, grouted securely in place. They will be placed in the structure so as to resist compressive loading, as in an axially-loaded or eccentrically-loaded bearing wall, or to resist lateral loads as a wall panel whose deformation is constrained by its framing beams and columns[15, 16]. The yield strength for the reinforced block is assumed to have a constant value of 1000 psi for both static and dynamic loading.

2.15 Structural Timber

The analytical equations of Chapter 3 are presented in general forms, and hence can be applied to all types of structural plywood and timber. The conventional working stresses for these materials, as listed in standard reference sources[17, 18, 19], can be converted directly to dynamic-loading yield stresses by applying a derived conversion factor. In obtaining this factor, the following influences were recognized[20].

(a) Effect of reducing load duration from permanent to blast duration is to increase conventional working stresses by a factor of 2.13.

(b) Effect of removing the assumed factor of safety is to increase conventional working stresses by a factor of 1.67.

(c) Effect of accepting some inherent variability in strength properties, thus acknowledging a probability of failure under design loading in some small percentage ($\leq$ 16%) of the members, is to increase the conventional working stresses by a factor of 1.13. This latter increase however, should not be applied to rolling shear in structural plywood.

(d) Net effect, excluding rolling shear in structural plywood, is to increase conventional working stresses in structural timber by $2.13 \times 1.67 \times 1.13 = 4.02$. A factor of four is therefore used to convert conventional timber working stresses to predicted yield stresses under dynamic loading.

2.16 Stabilized Earth

The earth which surrounds a buried shelter may itself be analyzed as a construction material of finite strength. Surface loading from an advancing blast wave must be transmitted through the soil to a buried structure. Hence, to whatever degree their strengths are simultaneously developed, the buried structure and the earth which surrounds it will act in unison to resist the applied loading. An exact interpretation of this interaction, presumably, would involve a detailed consideration of stresses and strains in the soil and of forces and displacements for the structure.

Unfortunately, while this general concept is reasonably clear, our knowledge of blast wave-soil-structure interaction is extremely limited. As a result, the current design practice[2, 3] is to attribute little or no resistance to the soil. It seems certain that such a simplification will, in many instances, result in an unknown degree of over-design for the structure. Limited experimental tests have been performed in this area, including some performed in connection with this study (see Appendix A), and all experimental results tend to confirm the importance of further investigations.

If it were possible to evaluate the importance of soil-strength in a soil-structure exposed to a blast environment, it would then become possible to evaluate the economic merits of soil stabilization. Such types of soil stabilization as increased compaction, mixtures of soils, chemical additives, addition of bituminous or Portland cement, electrolysis, etc. could be

compared in terms of cost, effectiveness, and permanence. In our present state of knowledge, however, it does not seem appropriate to supply definitive comments as to the economic desirability of soil stabilization in buried-shelter construction.

2.17 Fiber-Reinforced Plastics (FRP)

Plastic structural shapes, reinforced with glass fibers, are of growing interest for specialized structural applications. Their strength-to-weight ratios, in comparisons with other structural materials, are found to be favorable for selective uses. Their strength-to-cost ratios are less readily evaluated for comparison with better known structural materials, since an adequate basis for the projection of FRP in-place costs is lacking.

While the fiber-reinforced plastics are of structural interest and may be of eventual significance in buried-shelter construction, it is felt that the lack of definitive data relating to their structural behaviour and in-place cost should preclude their direct comparison with other structural materials. Accordingly, a discussion of fiber-reinforced plastics is included as Appendix B, rather than in the body of the report. The cost relationships which are discussed in this Appendix are relative, and are only valid when comparing the different types of reinforced plastic.

2.2 In-Place Unit Costs

2.21 Introduction

In-place cost, as explained in Section 1.3, is used as the basis for comparing structural materials in buried shelters. The validity of the cost data thus becomes of major importance. The costs used herein are representative of early 1963 prices in the Chicago Metropolitan Area, as obtained from material suppliers and various other sources[21, 22, 23, 24]. In this context, the term "in-place cost" is defined to include all necessary material, equipment and labor, as normally supplied by a general contractor when executing a competitively-awarded construction contract. The cost of the basic structural material is included, plus any fabrication costs, plus transportation and erection charges. To their cumulative total an additional 40% is added as an allowance for job overhead, general overhead, and profit.

The "in-place costs" thus include a provision for contractor's general overhead, job overhead, and profit. They do not, however, include any allowance for Architect-Engineer services in preparing preliminary designs, developing working drawings and bidding documents, and supplying general construction supervision. The unit costs also exclude site acquisition and preparation, charges incurred by various government agencies during the implementation and performance of the construction, or any contract expenses not directly allocable to the structural portion of a shelter.

2.22 Structural Steels

(a) Rolled Structural Shapes

Table 2-4

IN-PLACE COSTS OF ROLLED STRUCTURAL STEEL SHAPES, DOLLARS PER POUND (X_s = \$/lb)*

Dynamic Yield Stress, f_{dy}, psi	Wide-Flange Sections	I-Beam Sections
44,000	0.183	0.188
52,000	0.199	0.204
60,000	0.202	0.207

(b) Uniform-Thickness Plate, Curved

Table 2-5

IN-PLACE COSTS OF UNIFORM-THICKNESS CURVED STEEL PLATE, DOLLARS PER SQUARE FOOT OF CURVED SURFACE (X_s = \$/sq ft)

Thickness of Steel Plate, in.	Singly - Curved Plate			Doubly-Curved Plate		
	f_{dy} = 44,000 psi	f_{dy} 60,000 psi	f_{dy} 100,000 psi	f_{dy} 44,000 psi	f_{dy} 60,000 psi	f_{dy} 100,000 psi
1.00	14.35	15.75	20.60	20.00	21.65	25.70
0.75	10.20	11.25	14.45	13.37	14.60	18.20
0.50	6.08	6.70	8.90	8.22	8.95	11.40
0.25	3.21	3.53	4.45	4.45	4.84	5.90

* See page 3-3 for description of (X_n) cost factor notation

(c) Corrugated Steel Plate, Single-Curvature

Table 2-6

IN-PLACE COST OF SINGLE-CURVATURE CORRUGATED STEEL PLATE, DOLLARS PER SQUARE FOOT OF CURVED SURFACE (X_s = \$/sq ft)

Gage No.	f_{dy}= 44,000 psi	f_{dy}= 60,000 psi
12	2.84	3.30
10	2.94	3.43
8	3.18	3.70
7	3.33	3.88
5	3.75	4.40
3	4.10	4.80
1	4.32	5.05

2.23 Steel Reinforcing Rod

Table 2-7

IN-PLACE COST OF STEEL REINFORCING ROD, DOLLARS PER CUBIC FOOT OF STEEL (X_s = \$/ft^3)

Structural Elements	Flexural and Temperature Steel		Shear Reinforcement (Vertical Stirrups)	
	f_{dy}= 44,000 to 60,000 psi	f_{dy}= 75,000 psi	f_{dy}= 44,000 to 60,000 psi	f_{dy}= 75,000 psi
Slabs, beams, columns, walls, foundations	78.8	85.8	92.5	100.5
Shells	85.8	100.5	Not Applicable	Not Applicable

2.24 Structural Concrete

The estimated in-place costs for structural concrete are based on a ready-mix concrete which is hauled within the radius of the Chicago Metropolitan Area. If bucket placement is required for all structural elements, the costs shown for chuted concrete should be increased by five cents per cubic foot.

Table 2-8

IN-PLACE COST OF READY-MIX CONCRETE
DOLLARS PER CUBIC FOOT OF CONCRETE ($X_c = \$/ft^3$)

Ultimate Static Strength of Concrete, f'_c, psi*	Chute Placed		Bucket Placed
	Slabs and Beams	Foundation	Walls, Columns and Shells
2000	1.09	0.95	1.00
3000	1.14	1.00	1.05
4000	1.21	1.08	1.13
5000	1.29	1.16	1.21
6000	1.37	1.25	1.30

*Note that walls, columns and shells are designed on the basis of the dynamic ultimate strength f'_{dc}, rather than the static ultimate strength f'_c. The conversion is $f'_{dc} = 1.25 f'_c$ for all concrete strengths.

2.25 Concrete Forms

The unit costs for forms, as indicated in Table 2-9, are based on a minimum of two uses of the form material. Form work costs, at their best, are still the least dependable of the values quoted in this section. The cost of form work may vary more than 200% on identical structures, depending on the contractor's ingenuity and ability to organize and supervise this phase of construction.

Table 2-9

IN-PLACE COST OF CONCRETE FORMS, DOLLARS PER SQUARE FOOT OF CONCRETE SURFACE (X_f = \$/sq ft)

Type of Use	Cost of Form Work Dollars per Square Foot
Slabs and Beams	0.88
Walls and Rectangular Columns	1.00
Circular Columns	1.10
Shells -	
Barrel Arch	1.05
Domes and Cylinders	1.40
Sphere	1.75
Foundations	0.75
Slab Poured on Ground	0.60

2.26 Reinforced-Concrete Masonry Units

Table 2-10

IN-PLACE COST OF REINFORCED CONCRETE MASONRY UNITS, DOLLARS PER SQUARE FOOT OF WALL SURFACE (X_{cm} = \$/sq ft)

Unit	Cost - Dollars Per Square Foot
4" RCMU	1.01
6" RCMU	1.10
8" RCMU	1.20
12" RCMU	1.48

2.27 Structural Timber

Timber unit costs are derived for Southern Pine and/or Douglas Fir, since these two species are representative of commonly-available structural timbers. The unit cost includes freight from the mill to the Chicago Metropolitan Area, plus the cost of unloading and trucking to the job site. Since it is anticipated that the larger sizes will be required, a basic cost of \$130 per thousand board feet is assumed. An additional \$15/MBF is added to this, as a surcharge provision for shipments of less than a full carload lot. Since the shelter will be buried, an allowance of \$60/MBF was added for lumber treatment. Anticipated labor costs, based on a carpenter and helper, will amount to \$45/MBF. Adding these costs and including the 40% allowance for overhead and profit, a total unit cost, X_w, of \$350/MBF is established for either Southern Pine or Douglas Fir.

2.28 Earthwork

It is assumed that open-cut excavation, with 1:1 side slopes, will be accomplished by scrapers and tractor units. If conditions are such that shovel excavation is required, the unit costs listed herein should be increased by 40%.

Table 2-11

IN-PLACE COST OF EARTHWORK, DOLLARS PER CUBIC FOOT (X_e = \$/ft^3)

Earthwork Item	In-Place Cost Dollars Per Cubic Foot
Excavation	0.036
Back Fill	0.033
Haul of Waste	0.026
Total	0.095

2.29 Miscellaneous

a. Stairs

Spiral - \$750 per floor

Conventional - \$600 per floor

b. Angle supports for interior slabs

Shape	Slab Thickness, in.	\$/ft
Curved	6	5.75
	5	5.50
	4	5.00
Straight	6	3.25
	5	3.00
	4	2.50

2.3 Material Availability

The feasible scope for any mass shelter building program will, to a considerable extent, be dependent upon the availability of suitable structural materials. A historical record of the production and capacity of the major construction materials, with short-period projections of their future production and capacity, is given by Figures 2-1 to 2-5[25, 26, 27, 28, 29, 30]. In order to obtain a comprehensive understanding of this problem, four levels of availability are postulated. The first level of availability assumes no restrictions on the civilian economy and employs only the projected unused capacity of the materials industry. It is further assumed that no restriction

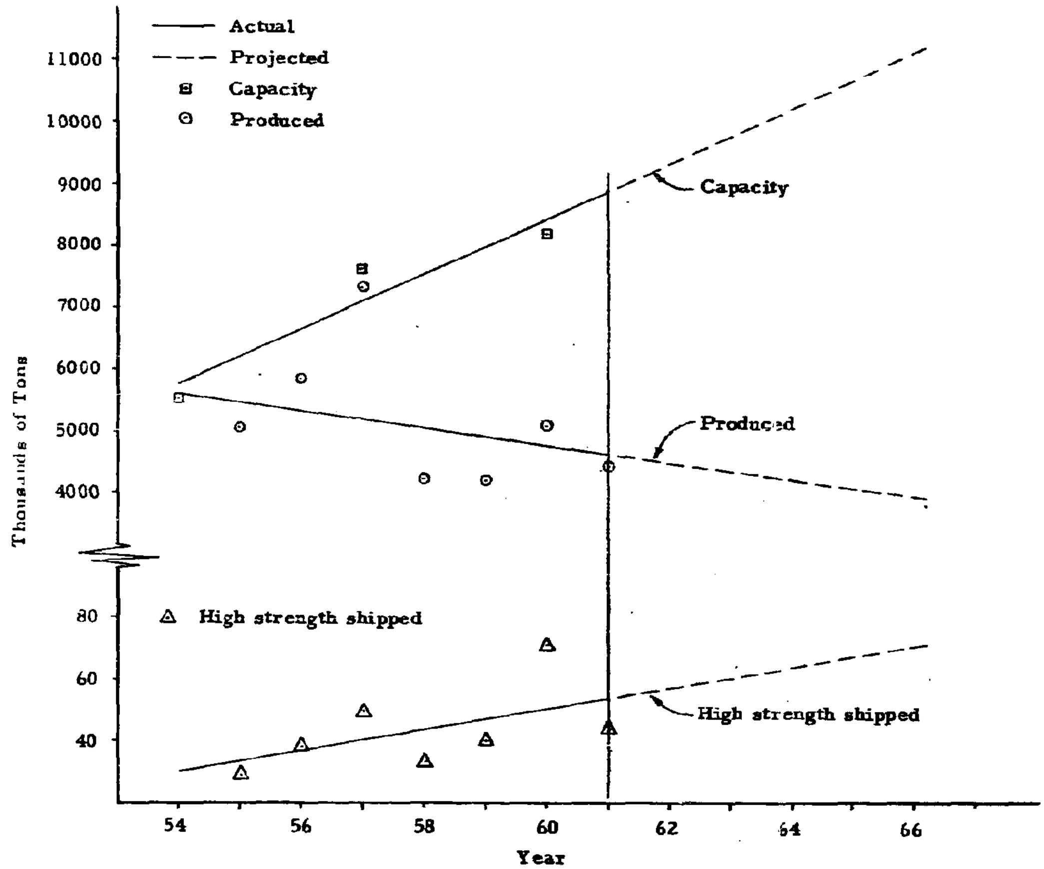


Figure 2-1

PRODUCTION AND CAPACITY OF HEAVY STRUCTURAL STEEL SHAPES

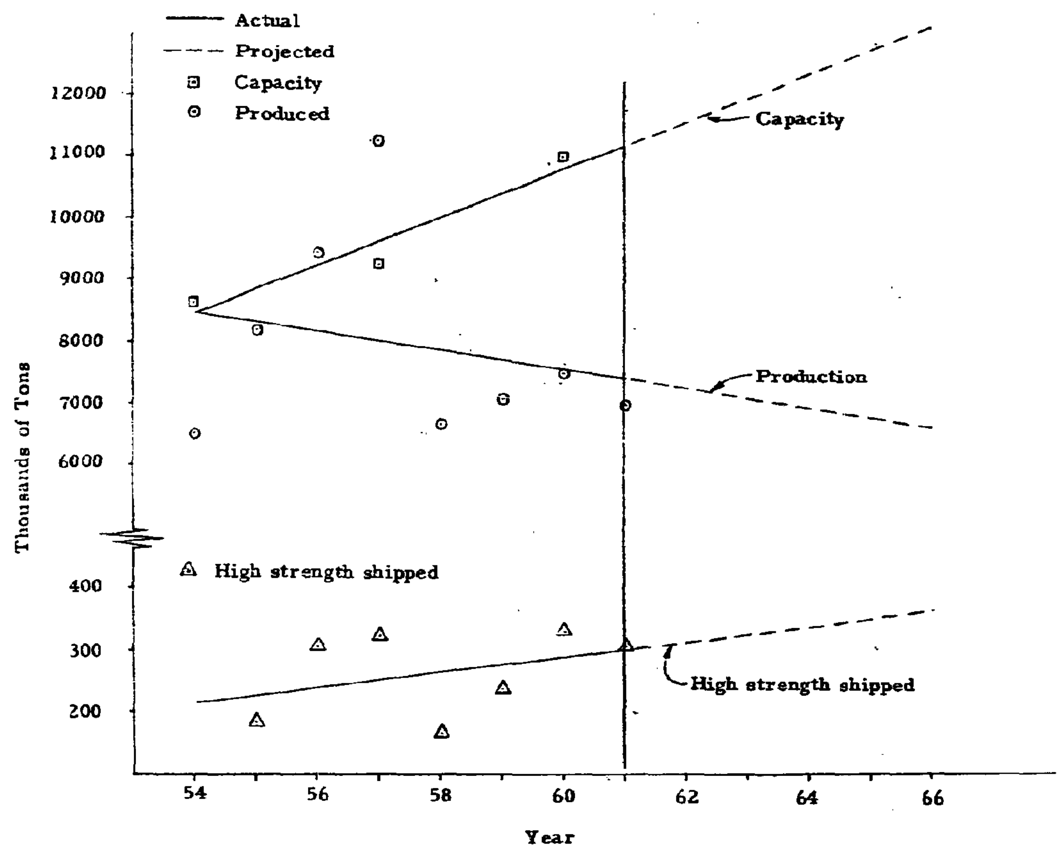


Figure 2-2
PRODUCTION AND CAPACITY OF STEEL PLATE

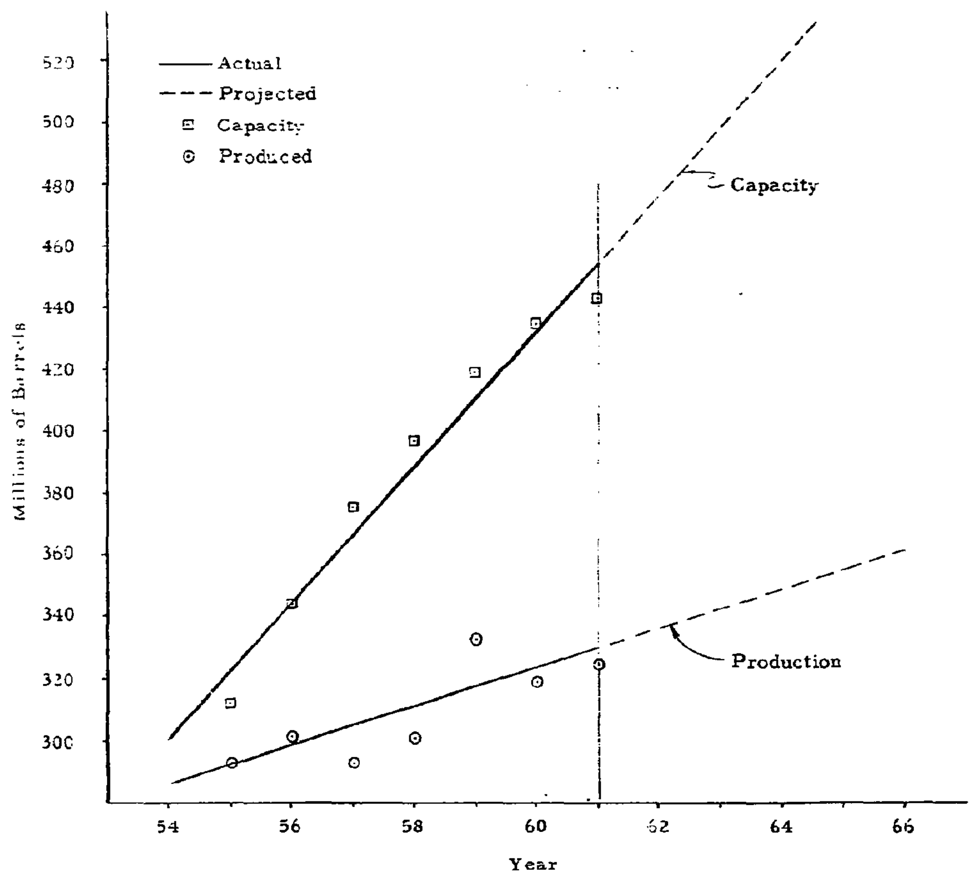


Figure 2-3
PRODUCTION AND CAPACITY OF PORTLAND CEMENT

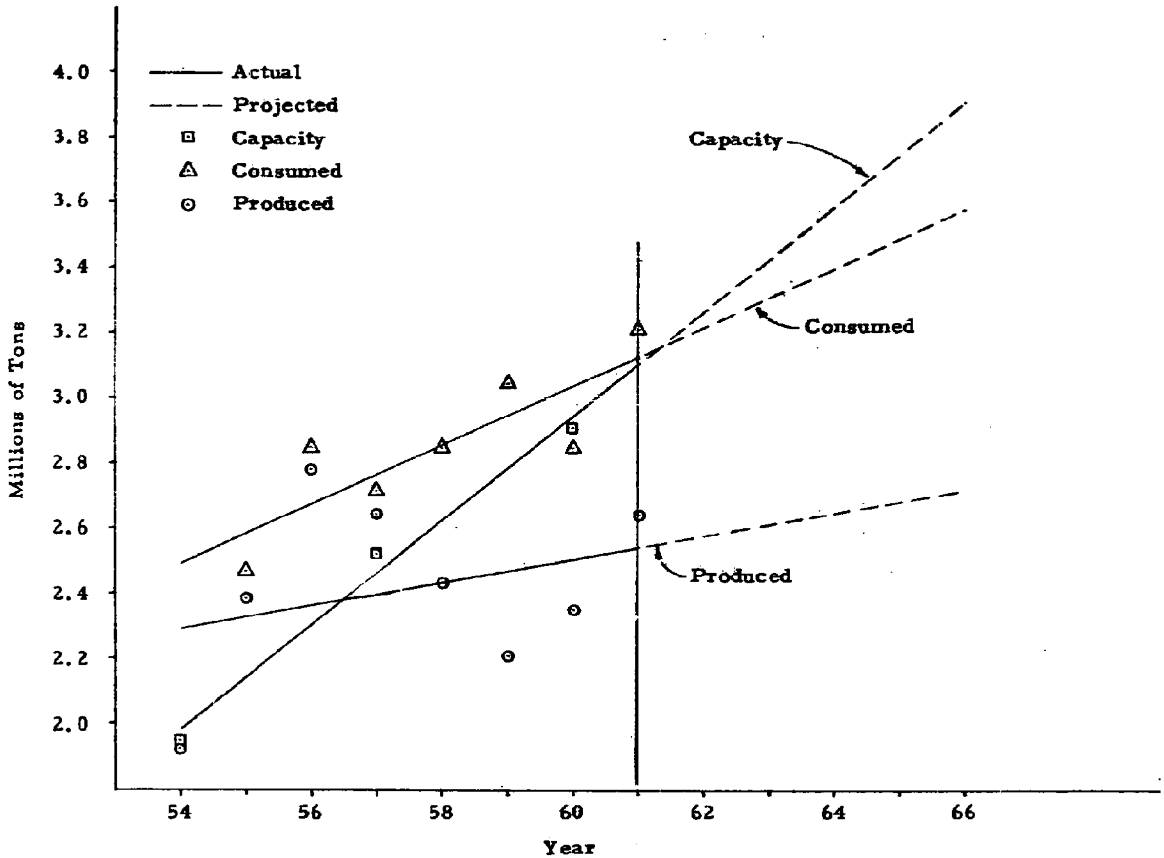


Figure 2-4
CAPACITY, PRODUCTION, AND CONSUMPTION OF STEEL REINFORCING BARS

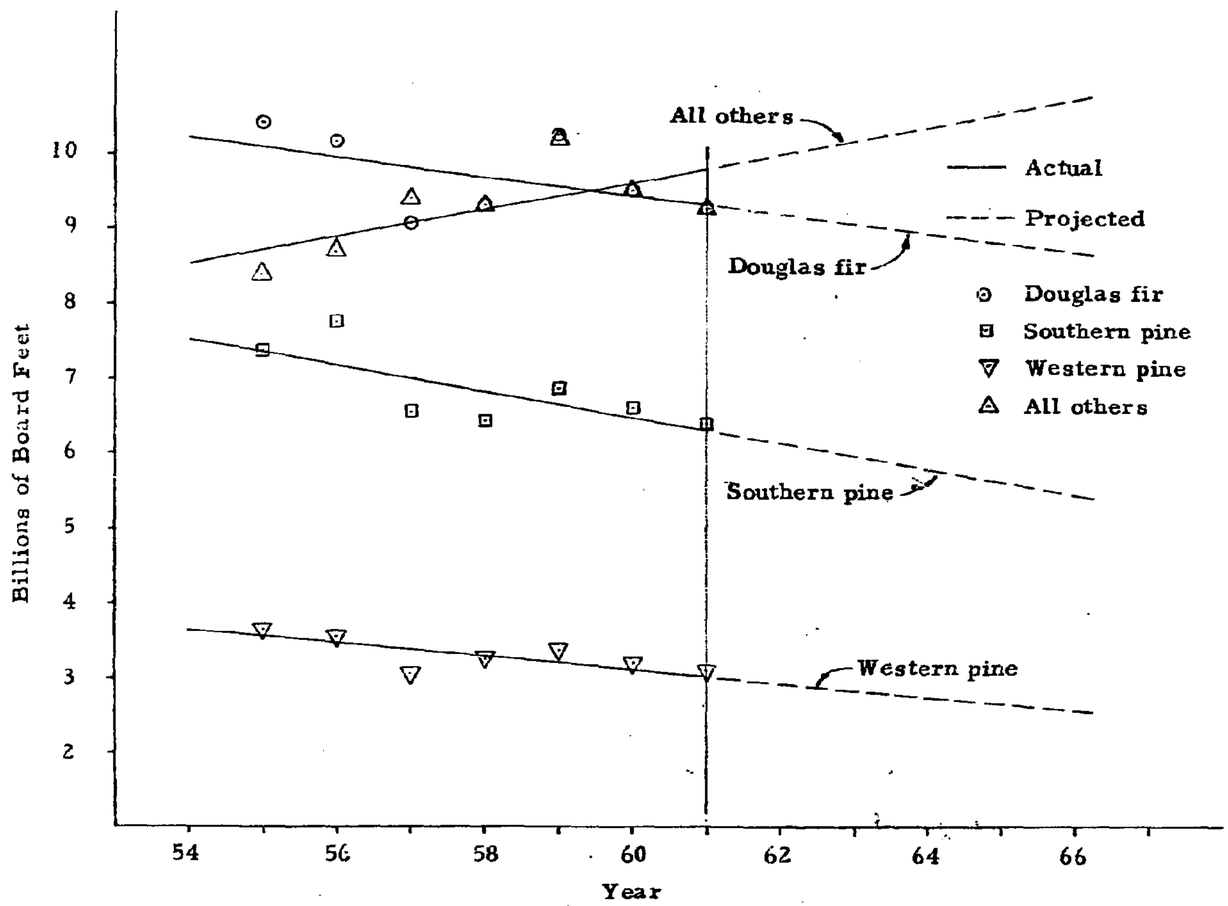


Figure 2-5

WOOD PRODUCTS - LUMBER PRODUCTION

is placed on foreign imports. The second level assumes a 10% restriction on the civilian economy, with no restriction on foreign imports. The third level again assumes a 10% reduction in material available to the civilian economy and, in addition, excludes foreign imports. The fourth level imposes a 25% restriction on the civilian economy and again excludes foreign imports.

These postulated conditions of availability are transformed into possible shelter construction programs by extrapolating the material requirements for a 100-man shelter which is designed to withstand an equivalent static pressure of 100 psi. This particular shelter, which is found to be the most economical in the 100-psi pressure range (Chapter 4), consists of a reinforced concrete cubicle with interior bearing walls. The basic material requirements for this shelter, assuming the most economical combinations of materials, are approximately 284 barrels of cement and 5.5 tons of reinforcing bar. When this shelter design is extrapolated for use in a mass shelter program, an analysis of production and capacity statistics indicates that reinforcing steel is the limiting material. It is interesting that reinforcing bar is the only major building material which is imported by this country in significant quantities.

Table 2-12

NATIONAL CAPABILITY FOR CONSTRUCTION OF 100-MAN, 100-PSI REINFORCED CONCRETE SHELTERS, BASED ON AVAILABILITY OF STEEL REINFORCING ROD

Postulated Material Availability	Construction Capability (Millions of Persons)				
	1964	1965	1966	1967	1968
No domestic restrictions, including imports	17.3	19.5	21.8	24.0	26.2
10% domestic restrictions, including imports	23.5	25.9	28.4	30.7	33.0
10% domestic restrictions, excluding imports	9.8	11.1	12.4	13.6	14.7
25% domestic restrictions, excluding imports	19.1	20.7	22.2	23.6	25.0

The surplus capacity of cement and form lumber is such that their domestic use need not be restricted, even assuming the larger programs for shelter construction.

Needless to say, should the occasion arise, the national capabilities for shelter construction which are indicated by Table 2-12 can be increased by substituting alternative designs and structural designs for the postulated minimum-cost shelter. For example, if steel shell structures are used to supplement the reinforced concrete cubicle, the projected capabities shown in Table 2-12 can be approximately doubled. These estimates of national capabilities for shelter construction are predicated on the assumption that the transportation, heavy equipment and construction industries could expand sufficiently to cope with the increased activity. The present state of the national economy makes this a reasonable assumption, assuming that military or civilian requirements of higher priority do not intervene.

CHAPTER 3 DESIGN OF STRUCTURAL ELEMENTS

3.1 Preliminary Considerations

The structural elements discussed in this chapter are designed as integral components of fully-buried, blast-resistant group shelters. These shelters, as discussed in Chapter 1, will be located above the ground water-table. The shelter configurations thus considered consist of the rectangular cubicle, the horizontal semi-circular arch or full cylinder, and the hemi-spherical dome or full sphere. Possible structural systems for such shelters range from unframed to fully framed construction. Thus, within feasible limits of shelter configuration and framing system, a given structural element may have a variety of functions.

A nuclear detonation will initiate a blast wave, propagating outward from the explosion center. Overpressure at the ground surface will be transmitted through the cover soil to a buried structure as a dynamic load. This load is assumed to act normal to all structural surfaces, whether plane or curved. The dynamic loading intensity for horizontal surfaces and for all curved surfaces is equated to the peak ground-level overpressure. A reduced loading intensity, related to characteristics of the adjacent soil, is assumed to act on vertical surfaces. For fully buried structures, such as are considered in this study, lateral soil restraint is considered to be sufficient to prevent buckling failures in compressive members.

The actual response of a structural element to a dynamically-applied load is influenced by material and load parameters. Many structural materials exhibit an increasing yield strength as the rate of load application is increased. For this reason, appropriate values for dynamic yield strength are introduced into the analyses of structural elements for a buried shelter. Also, if the dynamic loading is removed before a structural element has reached equi-librium under its action, the structural requirements may be less severe than for a longer duration of the same load. A detailed evaluation of structure response must consider the pressure-time variation of the applied loading and the frequency response of the structural element. In order to preserve

generality in the structural analyses, therefore, the elements have been designed for an "equivalent" uniform static loading rather than for a particular dynamic loading. This equivalent loading may subsequently be equated to an actual dynamic loading, at the discretion of the designer, by relating the weapon characteristics and the frequency response of the structural element[3, 7].

The buried shelter must retain its structural integrity throughout its period of projected use. This requirement may be of particular significance in the design of buried structures if a "negative" phase follows the positive shock loading. Full or partial reversal of stresses may result, and should be recognized in the structural analyses. The design should also provide for anticipated strain discontinuities and concentrations of secondary stresses. If these possibilities are recognized, careful design of structural details should preclude excessive spalling of concrete surfaces or similar evidences of localized structural distress. Welded joints for steel members are considered desirable, and customary anchorage requirements are supplemented by requiring that lapped reinforcing steel be welded[1, 3]. The design detailing of structural members should ensure, as a consistent requirement, that the full strength of the weakest member will be developed prior to failure of a structural connection.

Many of the details which must be given recognition in the actual design of a shelter are not explicitly considered in the following analyses of structural elements. Minimum thicknesses are specified for major structural members, however, as are minimum values for reinforcing steel in concrete elements. The estimated costs of structural details, where not explicitly appearing in the cost equations, are included in the unit in-place costs for the structural materials.

Cost expressions and equations which incorporate the various assumptions as to "in-place" costs are supplied for the several structural elements. These cost relationships may be adjusted for regional or secular variations in cost, if so desired, by substituting revised data into the basic cost expressions. Such a procedure is recommended if detailed cost studies are required for a specific shelter.

The generalized terms which appear in the cost equations of this and subsequent sections are expressed in several different ways. The

in-place cost of a material unit is first identified. The cost for a structural element is next computed as the linear sum of its component material costs. Finally, depending on the physical form of a structural element, its composite cost may be expressed as a total cost. In order to avoid any later confusion, the cost notations and their meaning are described as follows:

(1) $X_{(n)}$

This notation, characterized by an upper case X with lower case subscripts, refers to the in-place cost of a unit of a given material. It is expressed in dollars per unit area, (sq ft), unit weight, (lb), or unit volume, (cu ft), of the material.

(2) $C_{(n)}$

This notation, characterized by an upper case C with a lower case subscript, refers to the in-place material cost of a unit of a particular structural element. It is expressed in dollars per unit area, (sq ft) in the case of slabs, walls and shells, and in dollars per linear foot for column and beams.

(3) $C_{(N)}$

This notation, characterized by an upper case C and upper case subscript, refers to the total dollar cost of the materials in a structural element.

(4) k'_f

In specific instances where the unit cost, $X_{(n)}$ is dependent on some dimensional properties of the element, the $X_{(n)}$ term is replaced by the appropriate k term.

The material unit costs which are presented in Chapter 2 are used as the basis for determining element costs. A balanced approach to the application of cost data is achieved, where possible, by formulating cost equations which contain a sufficient number of terms for an adequate representation of the cost variables. The detail with which costs are investigated in this study is, to a considerable degree, dependent upon the probable importance of each element in over-all shelter design.

3.2 Structural Steel

3.21 Introduction

The structural steels considered in this study may be classified as plates or heavy structural shapes. The heavy structural shapes include those rolled beam and column sections which are available as standard items from steel suppliers. These sections can function as discrete structural elements in a framing system, or can be combined to form rectangular and segmented bents. Rolled plate and corrugated plate can be used as the load-supporting elements of singly and doubly curved surfaces, such as occur in cylinders and spheres. Because of the myriad combinations which become possible, this study will not include an investigation of built-up sections. Yield strength and primary failure modes, rather than deflection or stability criteria, are considered to govern the design of the steel elements. Detailed functional and design analyses for the various elements are contained in Sections 3.22 through 3.27.

The standard properties of rolled sections are fully described elsewhere[34, 35]. For convenience, however, the section properties of primary interest in buried shelter design are summarized in Table 3-1. Some latitude is available to the designer, since most of these rolled shapes can be obtained in different strength grades. Various high-strength steels, many with only slightly different physical properties, are available from the various steel suppliers (see Table 2-1). Approximately one percent of heavy structural shapes and five percent of plate steels are currently produced from high strength steels. The dynamic yield-stress values which are postulated for representative structural steels are listed in Chapter 2 as Table 2-2.

Cost data and equations for simplified costing of rolled beams are presented in Section 3.23. The costs for other steel elements, if required, can be based upon the weight of the member and the in-place unit costs supplied in Chapter 2. Structural steel, for the specialized applications considered in this study, faces three economic handicaps.

1) Corrosion protection of buried steel structures introduces problems which are outside the immediate sphere of structural design, but are directly related to the long-term strength properties of the steel section.

Table 3-1
PROPERTIES OF STANDARD ROLLED STEEL SHAPES

	Shape	Z $in.^3$	Z_r $in.^3$	Area $in.^2$	D in.	d_w in.	b in.	t_f in.	t_w in.	b/t_f	d_w/t_w	r_{gx} in.	r_{gy} in.
1.	36 WF 300	1105.1	1255.0	88.17	36.72	33.36	16.655	1.680	0.945	9.9	35.3	15.17	3.73
2.	36 WF 280	1031.2	1167.0	82.32	36.50	33.36	16.595	1.570	0.885	10.6	37.7	15.12	3.70
3.	36 WF 260	951.1	1076.0	76.56	36.24	33.36	16.555	1.440	0.845	11.5	39.5	15.00	3.65
4.	36 WF 245	892.5	1008.0	72.03	36.06	33.36	16.512	1.350	0.802	12.2	41.6	14.95	3.62
5.	36 WF 230	835.5	942.7	67.73	35.88	33.36	16.475	1.260	0.765	13.1	43.6	14.88	3.59
6.	36 WF 194	663.6	767.2	57.11	36.48	33.96	12.117	1.260	0.770	9.6	44.1	14.56	2.49
7.	36 WF 182	621.2	716.9	53.54	36.32	33.96	12.072	1.180	0.725	10.2	46.9	14.52	2.47
8.	36 WF 170	579.1	666.7	49.98	36.16	33.96	12.027	1.100	0.680	10.9	50.0	14.47	2.45
9.	36 WF 160	541.0	623.3	47.09	36.00	33.96	12.000	1.020	0.653	11.8	52.0	14.38	2.42
10.	36 WF 150	502.9	579.8	44.16	35.84	33.96	11.972	0.940	0.625	12.7	54.3	14.29	2.38
11.	36 WF 135	438.6	509.1	39.70	35.55	33.96	11.945	0.794	0.598	15.1	56.8	14.01	2.28
12.	33 WF 240	811.1	918.2	70.52	33.50	30.70	15.865	1.400	0.830	11.3	37.0	13.88	3.52
13.	33 WF 220	740.6	836.2	64.73	33.25	30.70	15.810	1.275	0.775	12.4	39.6	13.79	3.48
14.	33 WF 200	669.6	754.4	58.79	33.00	30.70	15.750	1.150	0.715	13.7	43.0	13.71	3.43
15.	33 WF 152	486.4	558.3	44.71	33.50	31.39	11.565	1.055	0.635	11.0	49.5	13.50	2.39
16.	33 WF 141	446.8	513.2	41.51	33.31	31.39	11.535	0.960	0.605	12.0	52.0	13.39	2.35
17.	33 WF 130	404.8	466.0	38.26	33.10	31.39	11.510	0.855	0.580	13.5	54.1	13.23	2.29
18.	33 WF 118	358.3	414.2	34.71	32.86	31.38	11.484	0.738	0.554	5.6	56.7	13.02	2.22
19.	30 WF 210	649.9	733.9	61.78	30.38	30.75	15.105	1.315	0.775	11.5	39.7	12.64	3.38
20.	30 WF 190	586.1	659.6	55.90	30.12	27.75	15.040	1.185	0.710	12.7	39.1	12.57	3.34
21.	30 WF 172	528.2	593.0	50.65	29.88	27.75	14.985	1.065	0.655	14.1	42.4	12.48	3.30
22.	30 WF 132	379.7	436.7	38.83	30.30	28.30	10.551	1.000	0.615	10.6	46.0	12.17	2.18
23.	30 WF 124	354.6	407.4	36.45	30.16	28.30	10.521	0.930	0.585	11.3	48.4	12.11	2.16
24.	30 WF 116	327.9	377.6	34.13	30.00	28.30	10.500	0.850	0.564	12.4	50.2	12.00	2.12
25.	30 WF 108	299.2	345.5	31.77	29.82	28.30	10.484	0.760	0.548	13.8	51.6	11.85	2.06
26.	30 WF 99	269.1	312.0	29.11	29.64	28.30	10.458	0.670	0.522	15.6	54.2	11.70	2.00
27.	27 WF 177	492.8	556.9	52.10	27.31	24.93	14.090	1.190	0.725	11.8	34.4	11.36	3.16
28.	27 WF 160	444.5	504.3	47.04	27.08	24.93	14.023	1.075	0.658	13.0	37.9	11.31	3.12
29.	27 WF 145	402.9	452.0	42.68	26.88	24.93	13.965	0.975	0.600	14.3	41.5	11.26	3.09
30.	27 WF 114	299.2	342.8	33.53	27.28	25.42	10.070	0.932	0.570	10.8	44.6	11.03	2.11
31.	27 WF 102	266.3	304.4	30.01	27.07	25.42	10.018	0.827	0.518	12.1	49.1	10.96	2.08
32.	27 WF 94	242.8	277.7	27.65	26.91	25.42	9.990	0.747	0.490	13.4	51.9	10.87	2.04
33.	27 WF 84	211.7	242.9	24.71	26.69	25.42	9.963	0.636	0.463	15.7	55.0	10.69	1.97
34.	24 WF 160	413.5	463.7	47.04	24.72	22.45	14.091	1.135	0.656	12.4	34.2	10.42	3.23
35.	24 WF 145	372.5	416.0	42.62	24.49	22.45	14.043	1.020	0.608	13.8	37.0	10.34	3.19
36.	24 WF 130	330.7	369.2	38.21	24.25	22.45	14.000	0.900	0.565	15.6	39.7	10.24	3.13
37.	24 WF 120	299.1	336.6	35.29	24.31	22.45	12.088	0.930	0.556	13.0	40.3	10.15	2.68
38.	24 WF 110	274.4	307.7	32.36	24.16	22.45	12.042	0.855	0.510	14.1	44.0	10.12	2.66
39.	24 WF 100	248.9	278.3	29.43	24.00	22.45	12.000	0.775	0.468	15.5	48.0	10.08	2.63
40.	24 WF 94	220.9	253.0	27.63	24.29	22.55	9.061	0.872	0.516	10.4	43.7	9.85	1.92
41.	24 WF 84	196.3	224.0	24.71	24.09	22.55	9.015	0.772	0.470	11.7	48.0	9.78	1.89
42.	24 WF 76	175.4	200.1	22.37	23.91	22.55	8.985	0.682	0.440	13.2	51.2	9.68	1.85
43.	24 WF 68	153.1	175.3	20.00	23.71	22.55	8.961	0.582	0.416	15.4	54.2	9.53	1.79
44.	21 WF 142	317.2	357.0	41.76	21.46	19.27	13.132	1.095	0.659	12.0	29.3	9.03	3.04

Table 3-1 (Continued)

PROPERTIES OF STANDARD ROLLED STEEL SHAPES

	Shape	Z in.^{3}	Z_r in.^{3}	Area in.^{2}	D in.	d_w in.	b in.	t_f in.	t_w in.	b/t_f	d_w/t_w	r_{gx} in.	r_{gy} in.
45.	21 WF 127	284.1	317.8	37.34	21.24	19.27	13.061	0.985	0.588	13.3	32.8	8.99	3.01
46.	21 WF 112	249.6	278.0	32.93	21.00	19.27	13.000	0.865	0.527	15.0	36.5	8.92	2.96
47.	21 WF 96	197.6	226.3	28.21	21.14	19.27	9.038	0.935	0.575	9.7	33.5	8.60	1.97
48.	21 WF 82	168.0	191.6	24.10	20.86	19.27	8.962	0.795	0.499	11.3	38.6	8.53	1.93
49.	21 WF 73	150.7	172.1	21.46	21.24	19.76	8.295	0.740	0.455	11.2	43.4	8.64	1.76
50.	21 WF 68	139.9	159.8	20.02	21.13	19.76	8.270	0.685	0.430	12.1	46.0	8.59	1.74
51.	21 WF 62	126.4	144.1	18.23	20.99	19.76	8.240	0.615	0.400	13.4	49.4	8.53	1.71
52.	21 WF 55	109.7	125.4	16.18	20.80	19.76	8.215	0.522	0.375	15.7	52.7	8.40	1.65
53.	18 WF 114	220.1	247.9	33.51	18.48	16.50	11.833	0.991	0.595	12.0	27.8	7.79	2.76
54.	18 WF 105	202.2	226.5	30.86	18.32	16.50	11.792	0.911	0.554	12.9	29.8	7.75	2.73
55.	18 WF 96	184.4	206.0	28.22	18.16	16.50	11.750	0.831	0.512	14.1	32.2	7.70	2.71
56.	18 WF 85	156.1	177.6	24.97	18.32	16.50	8.838	0.911	0.526	9.7	31.4	7.57	2.00
57.	18 WF 77	141.7	160.5	22.63	18.16	16.50	8.787	0.831	0.475	10.6	34.8	7.54	1.98
58.	18 WF 70	128.2	144.7	20.56	18.00	16.50	8.750	0.751	0.438	11.6	37.7	7.49	1.95
59.	18 WF 64	117.0	131.8	18.80	17.87	16.50	8.715	0.686	0.403	12.7	41.0	7.46	1.93
60.	18 WF 60	107.8	122.6	17.64	18.25	16.86	7.558	0.695	0.416	10.9	40.5	7.47	1.63
61.	18 WF 55	98.2	111.6	16.19	18.12	16.86	7.532	0.630	0.390	11.9	43.2	7.41	1.61
62.	18 WF 50	89.0	100.8	14.71	18.00	16.86	7.500	0.570	0.358	13.2	47.0	7.38	1.59
63.	18 WF 45	78.9	89.8	13.24	17.86	16.86	7.477	0.499	0.335	15.0	50.3	7.30	1.55
64.	16 WF 96	166.1	186.0	28.22	16.32	14.57	11.533	0.875	0.535	13.2	27.2	6.93	2.71
65.	16 WF 88	151.3	169.0	25.87	16.16	14.57	11.502	0.795	0.504	14.5	28.9	6.87	2.67
66.	16 WF 78	127.8	146.5	22.92	16.32	14.57	8.586	0.875	0.529	9.8	27.6	6.74	1.95
67.	16 WF 71	115.9	131.6	20.86	16.16	14.57	8.543	0.795	0.486	10.7	30.0	6.70	1.93
68.	16 WF 64	104.2	117.9	18.80	16.00	14.57	8.500	0.715	0.443	11.9	32.9	6.66	1.91
69.	16 WF 58	94.1	106.2	17.04	15.86	14.57	8.464	0.645	0.407	13.1	35.8	6.62	1.88
70.	16 WF 50	80.7	92.7	14.70	16.25	14.99	7.073	0.628	0.380	11.3	39.4	6.68	1.54
71.	16 WF 45	72.4	82.0	13.24	16.12	14.99	7.039	0.563	0.346	12.5	43.3	6.64	1.52
72.	16 WF 40	64.4	72.7	11.77	16.00	14.99	7.000	0.503	0.307	13.9	48.8	6.62	1.50
73.	16 WF 36	56.3	63.9	10.59	15.85	14.99	6.992	0.428	0.299	16.3	50.1	6.49	1.45
74.	14 WF 426	707.4	869.3	125.25	18.69	12.62	16.695	3.033	1.875	5.5	6.7	7.26	4.34
75.	14 WF 398	656.9	803.0	116.98	18.31	12.62	16.590	2.843	1.770	5.8	7.1	7.17	4.31
76.	14 WF 370	608.1	737.3	108.78	17.94	12.62	16.475	2.658	1.655	6.2	7.6	7.08	4.27
77.	14 WF 342	559.4		100.59	17.56	12.62	16.365	2.468	1.545	6.6	8.2	6.99	4.24
78.	14 WF 320	492.8	592.2	94.12	16.81	12.62	16.710	2.093	1.890	8.[illegible]	6.7	6.63	4.17
79.	14 WF 314	511.9	611.5	92.30	17.19	12.62	16.235	2.283	1.415	7.1	8.9	6.90	4.20
80.	14 WF 287	465.5	551.6	84.37	16.81	12.62	16.130	2.093	1.310	7.7	9.6	6.81	4.17
81.	14 WF 264	427.4	502.4	77.63	16.50	12.62	16.025	1.938	1.205	8.3	10.5	6.74	4.14
82.	14 WF 246	397.4	464.5	72.33	16.25	12.62	15.945	1.813	1.125	8.8	11.2	6.68	4.12
83.	14 WF 237	382.2	445.4	69.69	16.12	12.62	15.910	1.748	1.090	9.1	11.6	6.65	4.11
84.	14 WF 228	367.8	427.2	67.06	16.00	12.62	15.865	1.688	1.045	9.4	12.1	6.62	4.10
85.	14 WF 219	352.6	408.0	64.36	15.87	12.62	15.825	1.623	1.005	9.8	12.6	6.59	4.08
86.	14 WF 211	339.2	391.7	62.07	15.75	12.62	15.800	1.563	0.980	10.1	12.9	6.56	4.07
87.	14 WF 202	324.9	373.6	59.39	15.63	12.62	15.750	1.503	0.930	10.5	13.6	6.54	4.06
88.	14 WF 193	310.0	355.1	56.73	15.50	12.62	15.710	1.438	0.890	10.9	14.2	6.51	4.05
89.	14 WF 184	295.8	337.5	54.07	15.38	12.62	15.660	1.378	0.840	11.4	15.0	6.49	4.04
90.	14 WF 176	281.9	321.3	51.73	15.25	12.62	15.640	1.313	0.820	11.9	15.4	6.45	4.02

Table 3-1 (Continued)
PROPERTIES OF STANDARD ROLLED STEEL SHAPES

	Shape	Z in.^{3}	Z_r in.^{3}	Area in.^{2}	D in.	d_w in.	b in.	t_f in.	t_w in.	b/t_f	d_w/t_w	r_{gx} in.	r_{gy} in.
91.	14 WF 167	267.3	302.9	49.09	15.12	12.62	15.600	1.248	0.780	12.5	16.2	6.42	4.01
92.	14 WF 158	253.4	286.3	46.47	15.00	12.62	15.550	1.188	0.730	13.1	17.3	6.40	4.00
93.	14 WF 150	240.2	270.2	44.08	14.88	12.62	15.515	1.128	0.695	13.4	18.2	6.37	3.99
94.	14 WF 142	226.7	254.8	41.85	14.75	12.62	15.500	1.063	0.680	14.6	18.6	6.32	3.97
95.	14 WF 136	216.0	242.7	39.98	14.75	12.62	14.740	1.063	0.660	13.9	19.1	6.31	3.77
96.	14 WF 127	202.0	225.9	37.33	14.62	12.62	14.690	0.998	0.610	14.7	20.7	6.29	3.76
97.	14 WF 119	189.4	210.9	34.99	14.50	12.62	14.650	0.938	0.570	15.6	22.2	6.26	3.75
98.	14 WF 111	176.3	196.0	32.65	14.37	12.62	14.620	0.873	0.540	16.8	23.4	6.23	3.73
99.	14 WF 103	163.6	181.0	30.26	14.25	12.62	14.575	0.813	0.495	17.9	25.5	6.21	3.72
100.	14 WF 95	150.6	166.6	27.94	14.12	12.62	14.545	0.748	0.465	19.4	27.1	6.17	3.71
101.	14 WF 87	138.1	151.3	25.56	14.00	12.62	14.500	0.688	0.420	21.1	30.0	6.15	3.70
102.	14 WF 84	130.9	145.4	24.71	14.18	12.62	12.023	0.778	0.451	15.5	28.0	6.13	3.02
103.	14 WF 78	121.1	134.0	22.94	14.06	12.62	12.000	0.718	0.428	16.7	29.5	6.09	3.00
104.	14 WF 74	112.3	125.6	21.76	14.19	12.62	10.072	0.783	0.450	12.9	28.0	6.05	2.48
105.	14 WF 68	103.0	114.8	20.00	14.06	12.62	10.040	0.718	0.418	14.0	30.2	6.02	2.46
106.	14 WF 61	92.2	102.4	17.94	13.91	12.62	10.000	0.643	0.378	15.6	33.4	5.98	2.45
107.	14 WF 53	77.8	87.1	15.59	13.94	12.62	8.062	0.658	0.370	12.3	34.1	5.90	1.92
108.	14 WF 48	70.2	78.5	14.11	13.81	12.62	8.031	0.593	0.339	13.5	37.2	5.86	1.91
109.	14 WF 43	62.7	69.7	12.65	13.68	12.62	8.000	0.528	0.308	15.2	41.0	5.82	1.89
110.	14 WF 38	54.6	61.5	11.17	14.12	13.09	6.776	0.513	0.313	13.2	41.8	5.87	1.49
111.	14 WF 34	48.5	54.5	10.00	14.00	13.09	6.750	0.453	0.287	14.9	45.6	5.83	1.46
112.	14 WF 30	41.8	47.1	8.81	13.86	13.09	6.733	0.383	0.270	17.6	48.5	5.73	1.41
113.	12 WF 190	263.2	311.5	55.86	14.38	10.91	12.670	1.736	1.060	7.3	10.3	5.82	3.25
114.	12 WF 161	222.2	259.2	47.38	13.88	10.91	12.515	1.486	0.905	8.4	12.1	5.70	3.20
115.	12 WF 133	182.5	209.7	39.11	13.38	10.91	12.365	1.236	0.755	10.0	14.5	5.59	3.16
116.	12 WF 120	163.4	186.4	35.31	13.12	10.91	12.320	1.106	0.710	11.1	15.4	5.51	3.13
117.	12 WF 106	144.5	163.4	31.19	12.88	10.91	12.230	0.986	0.620	12.4	17.6	5.46	3.11
118.	12 WF 99	134.7	151.8	29.09	12.75	10.91	12.190	0.921	0.580	13.2	18.8	5.43	3.09
119.	12 WF 92	125.0	140.2	27.06	12.62	10.91	12.155	0.856	0.545	14.2	20.0	5.40	3.08
120.	12 WF 85	115.7	129.1	24.98	12.50	10.91	12.105	0.796	0.495	15.2	22.0	5.38	3.07
121.	12 WF 79	107.1	119.3	23.22	12.38	10.91	12.080	0.736	0.470	16.4	23.2	5.34	3.05
122.	12 WF 72	97.5	108.1	21.16	12.25	10.91	12.040	0.671	0.430	17.9	25.4	5.31	3.04
123.	12 WF 65	88.0	97.0	19.11	12.12	10.91	12.000	0.606	0.390	19.8	28.0	5.28	3.02
124.	12 WF 58	78.1	86.5	17.06	12.19	10.91	10.014	0.641	0.359	15.6	30.4	5.28	2.51
125.	12 WF 53	70.7	78.2	15.59	12.06	10.91	10.000	0.576	0.345	17.3	31.6	5.23	2.48
126.	12 WF 50	64.7	72.6	14.71	12.19	10.91	8.077	0.641	0.371	12.6	29.4	5.18	1.96
127.	12 WF 45	58.2	64.9	13.24	12.06	10.91	8.042	0.576	0.336	13.9	32.5	5.15	1.94
128.	12 WF 40	51.9	57.6	11.77	11.94	10.91	8.000	0.516	0.294	15.5	37.1	5.13	1.94
129.	12 WF 36	45.9	51.4	10.59	12.24	11.16	6.565	0.540	0.305	12.2	36.6	5.15	1.50
130.	12 WF 31	39.4	44.0	9.12	12.09	11.16	6.525	0.465	0.265	14.0	42.0	5.11	1.47
131.	12 WF 27	34.1	38.0	7.97	11.95	11.15	6.500	0.400	0.240	16.3	46.5	5.06	1.44
132.	10 WF 112	126.3	1[illegible]7.5	32.92	11.38	8.88	10.415	1.248	0.755	8.4	11.8	4.67	2.67
133.	10 WF 100	112.4	130.1	29.43	11.12	8.88	10.345	1.118	0.685	9.3	13.0	4.61	2.65
134.	10 WF 89	99.7	114.4	26.19	10.88	8.88	10.275	0.998	0.615	10.3	14.4	4.55	2.63
135.	10 WF 77	86.1	97.7	22.67	10.62	8.88	10.195	0.868	0.535	11.8	16.6	4.49	2.60
136.	10 WF 72	80.1	90.7	21.18	10.50	8.88	10.170	0.808	0.510	12.6	17.4	4.46	2.59

Table 3-1 (Continued)
PROPERTIES OF STANDARD ROLLED STEEL SHAPES

	Shape	Z in.³	Z_r in.³	Area in.²	D in.	d_w in.	b in.	t_f in.	t_w in.	b/t_f	d_w/t_w	r_{gx} in.	r_{gy} in.
137.	10 WF 66	73.7	82.8	19.41	10.38	8.88	10.117	0.748	0.457	13.5	19.4	4.44	2.58
138.	10 WF 60	67.1	75.1	17.66	10.25	8.88	10.075	0.683	0.415	14.8	21.4	4.41	2.57
139.	10 WF 54	60.4	67.0	15.88	10.12	8.88	10.028	0.618	0.368	16.2	24.1	4.39	2.56
140.	10 WF 49	54.6	60.3	14.40	10.00	8.88	10.000	0.558	0.340	17.9	26.1	4.35	2.54
141.	10 WF 45	49.1	55.0	13.24	10.12	8.88	8.022	0.618	0.350	13.0	25.4	4.33	2.00
142.	10 WF 39	42.2	47.0	11.48	9.94	8.88	7.990	0.528	0.318	15.1	27.9	4.27	1.98
143.	10 WF 33	35.0	38.8	9.71	9.75	8.88	7.964	0.433	0.292	18.4	30.4	4.20	1.94
144.	10 WF 29	30.8	34.7	8.53	10.22	9.22	5.799	0.500	0.289	11.6	31.9	4.29	1.34
145.	10 WF 25	26.4	29.5	7.35	10.08	9.22	5.762	0.430	0.252	13.4	36.6	4.26	1.31
146.	10 WF 21	21.5	24.1	6.19	9.90	9.22	5.750	0.340	0.240	16.9	38.4	4.14	1.25
147.	8 WF 67	60.4	70.1	19.70	9.00	7.13	8.287	0.933	0.575	8.9	12.4	3.71	2.12
148.	8 WF 58	52.0	59.9	17.06	8.75	7.13	8.222	0.808	0.510	10.2	14.0	3.65	2.10
149.	8 WF 48	43.2	49.0	14.11	8.50	7.13	8.117	0.683	0.405	11.9	17.6	3.61	2.08
150.	8 WF 40	35.5	39.9	11.76	8.25	7.13	8.077	0.558	0.365	14.5	19.5	3.53	2.04
151.	8 WF 35	31.1	34.7	10.30	8.12	7.13	8.027	0.493	0.315	16.3	22.6	3.50	2.03
152.	8 WF 31	27.4	30.4	9.12	8.00	7.13	8.000	0.433	0.288	18.5	24.8	3.47	2.01
153.	8 WF 28	24.3	27.1	8.23	8.06	7.13	6.540	0.463	0.285	14.1	25.0	3.45	1.62
154.	8 WF 24	20.8	23.1	7.06	7.93	7.13	6.500	0.398	0.245	16.3	29.1	3.42	1.61
155.	8 WF 20	17.0	19.1	5.88	8.14	7.38	5.268	0.378	0.248	13.9	29.8	3.43	1.20
156.	8 WF 17	14.1	15.8	5.00	8.00	7.38	5.250	0.308	0.230	17.0	32.1	3.36	1.16
157.	6 WF 25	16.8	19.0	7.37	6.37	5.46	6.080	0.456	0.320	13.3	17.1	2.69	1.52
158.	6 WF 20	13.4	15.0	5.90	6.20	5.47	6.018	0.367	0.258	16.4	21.2	2.66	1.50
159.	6 WF 15.5	10.1	11.3	4.62	6.00	5.46	6.000	0.269	0.240	22.3	22.8	2.56	1.45
160.	5 WF 18.5	9.94	11.4	5.45	5.12	4.28	5.025	0.420	0.265	12.0	16.2	2.16	1.28
161.	5 WF 16	8.53	9.6	4.70	5.00	4.28	5.000	0.360	0.240	13.9	17.8	2.13	1.26
162.	4 WF 13	5.45	6.3	3.82	4.16	3.47	4.060	0.345	0.280	11.8	12.4	1.72	0.99
163.	24 I 120	250.9	298.0	35.13	24.00	21.80	8.048	1.102	0.798	7.3	27.3	9.26	1.56
164.	24 I 105.9	234.3	273.0	30.98	24.00	21.80	7.875	1.102	0.625	7.2	34.9	9.53	1.60
165.	24 I 100	197.6	238.8	29.25	24.00	22.26	7.247	0.871	0.747	8.3	29.8	9.05	1.29
166.	24 I 90	185.8	220.5	26.30	24.00	22.26	7.124	0.871	0.624	8.2	35.7	9.21	1.32
167.	24 I 79.9	173.9	203.0	23.33	24.00	22.26	7.000	0.871	0.500	8.0	44.5	9.46	1.36
168.	20 I 95	160.0	192.0	27.74	20.00	18.17	7.200	0.916	0.800	7.9	22.7	7.59	1.35
169.	20 I 85	150.2	177.3	24.80	20.00	18.17	7.053	0.916	0.653	7.7	27.8	7.78	1.38
170.	20 I 75	126.3	151.5	21.90	20.00	18.42	6.391	0.789	0.641	8.1	28.8	7.60	1.17
171.	20 I 65.4	116.9	137.3	19.08	20.00	18.42	6.250	0.789	0.500	7.9	36.8	7.83	1.21
172.	18 I 70	101.9	123.8	20.46	18.00	16.62	6.251	0.691	0.711	9.0	23.4	6.70	1.09
173.	18 I 54.7	88.4	103.5	15.94	18.00	16.62	6.000	0.691	0.460	8.7	36.1	7.07	1.15
174.	15 I 50	64.2	76.5	14.59	15.00	13.76	5.640	0.622	0.550	9.1	25.0	5.74	1.05
175.	15 I 42.9	58.9	68.6	12.49	15.00	13.76	5.500	0.622	0.410	8.8	33.6	5.95	1.08
176.	12 I 50	50.3	60.7	14.57	12.00	10.68	5.477	0.659	0.687	8.3	15.5	4.55	1.05
177.	12 I 40.8	44.8	52.5	11.84	12.00	10.68	5.250	0.659	0.460	8.0	23.2	4.77	1.08
178.	12 I 35	37.8	44.4	10.20	12.00	10.91	5.078	0.544	0.428	9.3	25.5	4.72	0.99
179.	12 I 31.8	36.0	41.6	9.26	12.00	10.91	5.000	0.544	0.350	9.2	31.2	4.83	1.01
180.	10 I 35	29.2	35.2	10.22	10.00	9.02	4.944	0.491	0.594	10.1	15.2	3.78	0.91
181.	10 I 25.4	24.4	28.0	7.38	10.00	9.02	4.660	0.491	0.310	9.5	29.1	4.07	0.97
182.	8 I 23	16.0	19.2	6.71	8.00	7.15	4.171	0.425	0.441	9.8	16.2	3.09	0.81

Table 3-1 (Continued)

PROPERTIES OF STANDARD ROLLED STEEL SHAPES

	Shape		Z in.^{3}	Z_r in.^{3}	Area in.^{2}	D in.	d_w in.	b in.	t_f in.	t_w in.	b/t_f	d_w/t_w	r_{gx} in.	r_{gy} in.
183.	8 I	18.4	14.2	16.3	5.34	8.00	7.15	4.000	0.425	0.270	9.4	26.5	3.26	0.84
184.	7 I	20	12.0	14.4	5.83	7.00	6.22	3.860	0.392	0.450	9.9	13.8	2.68	0.74
185.	7 I	15.3	10.4	11.9	4.43	7.00	6.22	3.660	0.392	0.250	9.3	24.9	2.86	0.78
186.	6 I	17.25	8.7	10.5	5.02	6.00	5.28	3.565	0.359	0.465	9.9	11.4	2.28	0.68
187.	6 I	12.5	7.3	8.4	3.61	6.00	5.28	3.330	0.359	0.230	9.3	23.0	2.46	0.72
188.	5 I	14.75	6.0	7.4	4.29	5.00	4.35	3.284	0.326	0.494	10.1	8.8	1.87	0.63
189.	5 I	10	4.8	5.6	2.87	5.00	4.35	3.000	0.326	0.210	9.2	20.7	2.05	0.65
190.	4 I	9.5	3.3	4.0	2.76	4.00	3.41	2.796	0.293	0.326	9.6	10.5	1.56	0.58
191.	4 I	7.7	3.0	3.5	2.21	4.00	3.41	2.660	0.293	0.190	9.1	11.4	1.64	0.59
192.	3 I	7.5	1.9	2.3	2.17	3.00	2.48	2.509	0.260	0.349	9.7	7.1	1.15	0.52
193.	3 I	5.7	1.7	1.9	1.64	3.00	2.48	2.330	0.260	0.170	9.0	14.6	1.23	0.53
194.	16 B	31	47.0	53.9	9.12	15.84	14.96	5.525	0.442	0.275	12.5	54.4	6.39	1.13
195.	16 B	26	38.1	43.8	7.65	15.65	14.96	5.500	0.345	0.250	15.9	59.8	6.24	1.07
196.	14 B	26	34.9	39.8	7.65	13.89	13.05	5.025	0.418	0.255	12.0	51.2	5.63	1.04
197.	14 B	22	28.8	32.9	6.47	13.72	13.05	5.000	0.335	0.230	11.9	56.7	5.52	0.99
198.	14 B	17.2	21.0	24.4	5.05	14.00	13.46	4.000	0.272	0.210	14.7	64.0	5.40	0.72
199.	12 B	22	25.3	29.4	6.47	12.31	11.46	4.030	0.424	0.260	9.5	44.0	4.91	0.84
200.	12 B	19	21.4	24.8	5.62	12.16	11.46	4.010	0.349	0.240	11.5	47.7	4.81	0.81
201.	12 B	16.5	17.5	20.6	4.86	12.00	11.46	4.000	0.269	0.230	14.9	49.8	4.65	0.76
202.	12 B	14	14.8	17.4	4.14	11.91	11.46	3.970	0.224	0.200	17.7	57.3	4.61	0.74
203.	10 B	19	18.8	21.6	5.61	10.25	9.46	4.020	0.394	0.250	10.2	37.8	4.14	0.86
204.	10 B	17	16.2	18.6	4.98	10.12	9.46	4.010	0.329	0.240	12.2	39.4	4.05	0.83
205.	10 B	15	13.8	16.0	4.40	10.00	9.46	4.000	0.269	0.230	14.9	41.1	3.95	0.80
206.	10 B	11.5	10.5	12.1	3.39	9.87	9.46	3.950	0.204	0.180	19.4	52.6	3.92	0.77
207.	8 B	15	11.8	13.6	4.43	8.12	7.49	4.015	0.314	0.245	12.8	30.6	3.29	0.86
208.	8 B	13	9.88	11.4	3.83	8.00	7.49	4.000	0.254	0.230	15.7	32.5	3.21	0.83
209.	8 B	10	7.79	8.9	2.95	7.90	7.49	3.940	0.204	0.170	19.3	44.0	3.23	0.82
210.	6 B	16	10.1	11.6	4.72	6.25	5.44	4.030	0.404	0.260	10.0	20.9	2.59	0.96
211.	6 B	12	7.24	8.3	3.53	6.00	5.44	4.000	0.279	0.230	14.3	23.6	2.48	0.90
212.	6 B	8.5	5.07	5.7	2.50	5.83	5.44	3.940	0.194	0.170	20.3	32.0	2.43	0.87
213.	8 M	34.5	28.9	32.8	10.09	8.00	7.12	8.000	0.438	0.375	18.3	19.0	3.40	1.87
214.	8 M	24	21.0	23.4	7.06	8.00	7.25	6.50	0.375	0.24	17.3	30.2	3.45	1.53
215.	8 M	20	15.2	17.5	5.88	8.00	7.37	5.36	0.313	0.35	17.1	21.0	3.22	1.06
216.	8 M	17	14.0	15.7	5.00	8.00	7.38	5.25	0.312	0.24	16.8	30.8	3.35	1.11
217.	6 M	25	15.7	17.9	7.35	6.00	5.00	5.938	0.500	0.313	11.9	16.0	2.53	1.43
218.	6 M	20	12.9	14.6	5.88	6.00	5.25	5.938	0.375	0.250	15.8	21.0	2.57	1.39
219.	5 M	18.9	9.5	11.1	5.56	5.00	4.12	5.000	0.438	0.313	11.4	13.2	2.08	1.20
220.	4 M	13	5.2	6.1	3.82	4.00	3.25	3.937	0.375	0.250	10.5	13.0	1.65	0.94
221.	12 Jr	11.8	12.0	14.2	3.45	12.00	11.50	3.063	0.250	0.175	12.3	65.7	4.57	0.53
222.	10 Jr	9	7.8	9.2	2.64	10.00	9.62	2.688	0.188	0.155	14.3	62.0	3.85	0.48
223.	8 Jr	6.5	4.7	5.4	1.92	8.00	7.62	2.281	0.188	0.135	12.1	56.5	3.12	0.42
224.	7 Jr	5.5	3.5	4.0	1.61	7.00	6.62	2.078	0.188	0.126	11.0	52.5	2.74	0.39
225.	6 Jr	4.4	2.4	2.8	1.30	6.00	5.62	1.844	0.188	0.114	9.8	49.3	2.37	0.36

2) Most rolled shapes are proportioned for conventional structural applications, where moment rather than shear is critical. For the loads and spans considered in this study, however, shear almost invariably governs the selection of steel members. The available steel shapes are relatively inefficient in resisting large shearing forces, and a considerable portion of their theoretically-available moment capacity must thus remain unutilized.

3) A high modulus of elasticity and a favorable ratio of dead-weight to live-load capacity normally makes steel competitive as a construction material. However, these advantages are largely negated by the large live-loads and the short span, single story layouts which are associated with buried group shelters. The advantages still retained by steel include its ability to absorb large amounts of energy and to experience large plastic deformations before fracture. These elastic properties of steel, while perhaps giving the designer more confidence in the structural behavior of the material in a complex dynamic-blast environment, unfortunately do not appear to offset the inherent economic disadvantages.

3.22 Rolled Column Section

Steel columns have several possible applications in buried shelters. Most of these will occur in the cubicle configuration, where the column may function either as a vertical member in a steel framing system or as an isolated support for beams and/or decking.

Various equations have been proposed for the ultimate design of short steel columns[2, 3, 31, 32, 33, 34, 35]. It is frequently assumed that side sway will not be a problem in a buried blast-resistant shelter, and the full dynamic yield strength of the column can thus be developed prior to its failure. The equations for the ultimate dynamic strength of a steel column which are used in this study are derived from data supplied by Reference 32. These equations are applicable to rolled sections whose H/r_g ratios, expressed in the stated units, do not exceed five.

Axially-Loaded Column

$$P_{dy} = A\, f_{dy} \left(1 - \frac{\alpha H}{5000\, r_g} \sqrt{f_{dy}}\right) \qquad (3.22.1)$$

Eccentrically-Loaded Column

$$\frac{P'_{dy}}{A\, f_{dy}} = 1 - 0.85\, \frac{M'_p}{M_p} - \frac{\alpha H}{5000\, r_g} \sqrt{f_{dy}} \qquad (3.22.2)$$

where

P_{dy} = dynamic yield resistance in direct compression of an axially-loaded steel column, (lb)

P'_{dy} = dynamic yield resistance in direct compression of an eccentrically-loaded column, (lb)

A = gross area of column section, (sq in.)

f_{dy} = dynamic yield stress for steel in tension or compression, (psi)

α = effective length factor for column

H = unsupported column height, (ft)

M_p = full plastic moment capacity of column section, considered as a beam, (in.-lb)

M'_p = plastic moment which can be actually resisted by an eccentrically loaded column, (in.-lb)

r_g = radius of gyration of column section about assumed bending axis, (in.)

For illustration, Figure 3-1 compares values of P'_{dy}/P_{dy} and M'_p/M_p, as computed by Equation 3.22.1, with those proposed in other references(2, 3, 7) for similar applications. The in-place cost of a steel column can be estimated by multiplying its weight per foot by the product of the unit cost coefficient for structural steel, as listed in Chapter 2, and the length of the column in feet.

3.23 Rolled Beam Section

Rolled steel sections can be incorporated in an all-steel framing system or used as beams in a composite system. The shapes of the standard rolled sections have evolved from their use in conventional construction, where flexural stresses are usually controlling. These shapes are highly efficient for such applications, particularly if the design of the member is based upon elastic deflections. In buried shelters, however, the span lengths are small and the design loads can be very large. As a result, in almost every case examined in this study, the capacity of a rolled beam section is limited by its ability to resist shearing stresses.

The rolled beam sections are analyzed on the assumption that an equivalent uniform load is statically applied over the full length of the member. Possible modes of failure include flexure, shear, and local buckling. Flexure and shearing failures will be initiated by localized plastic yielding at critically-stressed sections, with a continuing redistribution of internal stresses until a yield mechanism has been developed. The yield resistances in flexure and in shear are expressed in terms of the maximum uniform loadings q_f, q_v, which can be supported by a given beam. Three possible conditions of end restraint are considered for each beam.

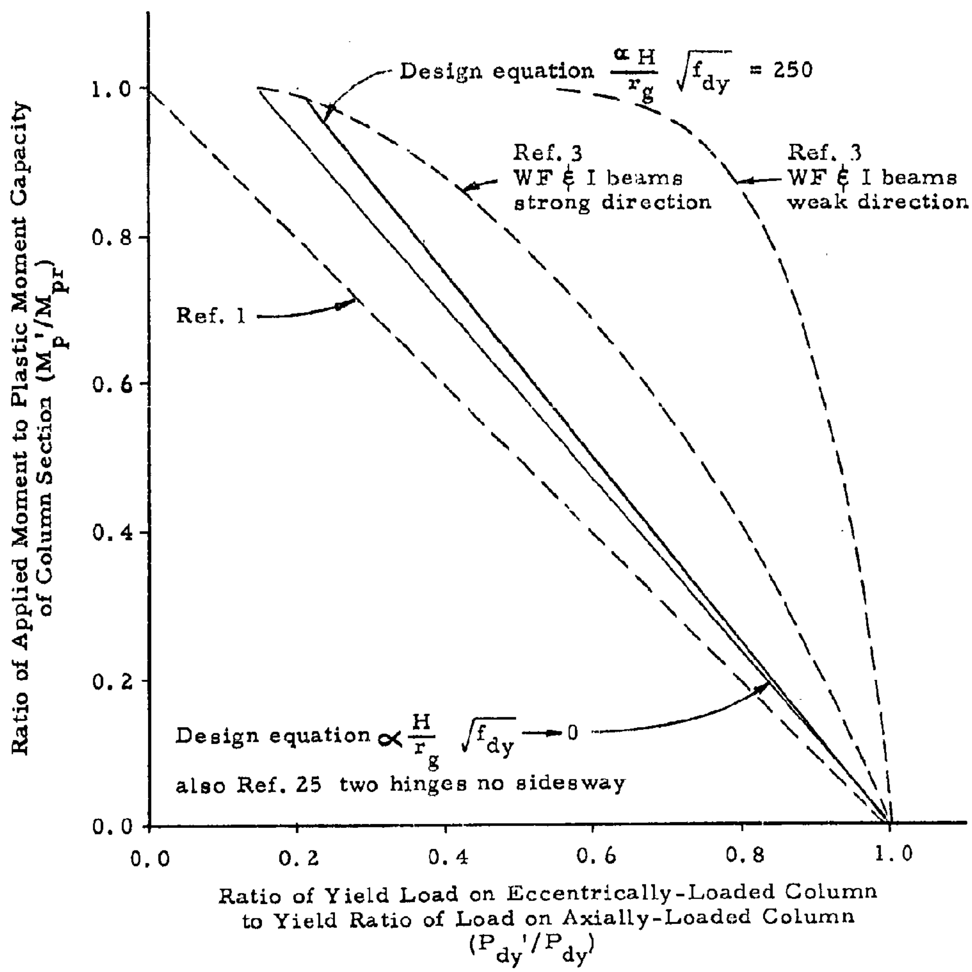


Figure 3-1

PREDICTIONS OF YIELD LOADS
FOR ECCENTRICALLY-LOADED SHORT STEEL COLUMNS

Peak flexural and shearing stresses can simultaneously occur on the critically-stressed sections of a loaded beam. It is assumed that only the web of a rolled section is effective in resisting shear, and that a steel fiber which is yielding in flexure will have no capacity to resist shear. Applying these assumptions to the section with the maximum ratio of moment to shear (M/V ratio), the web is considered to resist only the shear and the flanges are considered to resist only flexure. Accordingly, a reduced plastic moment capacity M_{pr}, based on the reduced plastic modulus Z_r of the flange areas, is used in lieu of the full plastic moment M_p in analyzing the moment resistance at such sections.

The relation between yield resistance and length of a beam can be studied by assuming that a short beam is loaded to incipient yield in shear. Its total resistance to load thus becomes a function of the sectional area of the web. As the length of the beam is increased, its total load resistance will remain unchanged as long as web shear continues to control. However, peak flexural stresses will increase rapidly and at some length, designated as L_{ep}, the extreme fibers will start to yield in flexure. As the length of beam continues to increase, with total load still constant, plastic hinges progressively develop at locations of maximum moment (elasto-plastic range). At some length, designated as L_{fv}, the beam with constant total load is simultaneously at plastic yielding in flexure and in shear. If the length of beam is increased still further, the total allowable load on the beam must be reduced since the plastic moment capacity (M_p or M_{pr}) will now control the total resistance of the beam. These general relationships are illustrated in Figure 3-2.

The yield resistances q_f and q_v for rolled steel beams supporting uniform equivalent loads can be computed as follows:

(1) Simply Supported Beams

$$q_f \times 12B = \frac{8M_p}{144L^2} = \frac{8Zf_{dy}}{144L^2}$$

$$q_v \times 12B = \frac{2V_p}{12L} = \frac{2v_{dy}A_w}{12L} = \frac{1.2A_w f_{dy}}{12L}$$

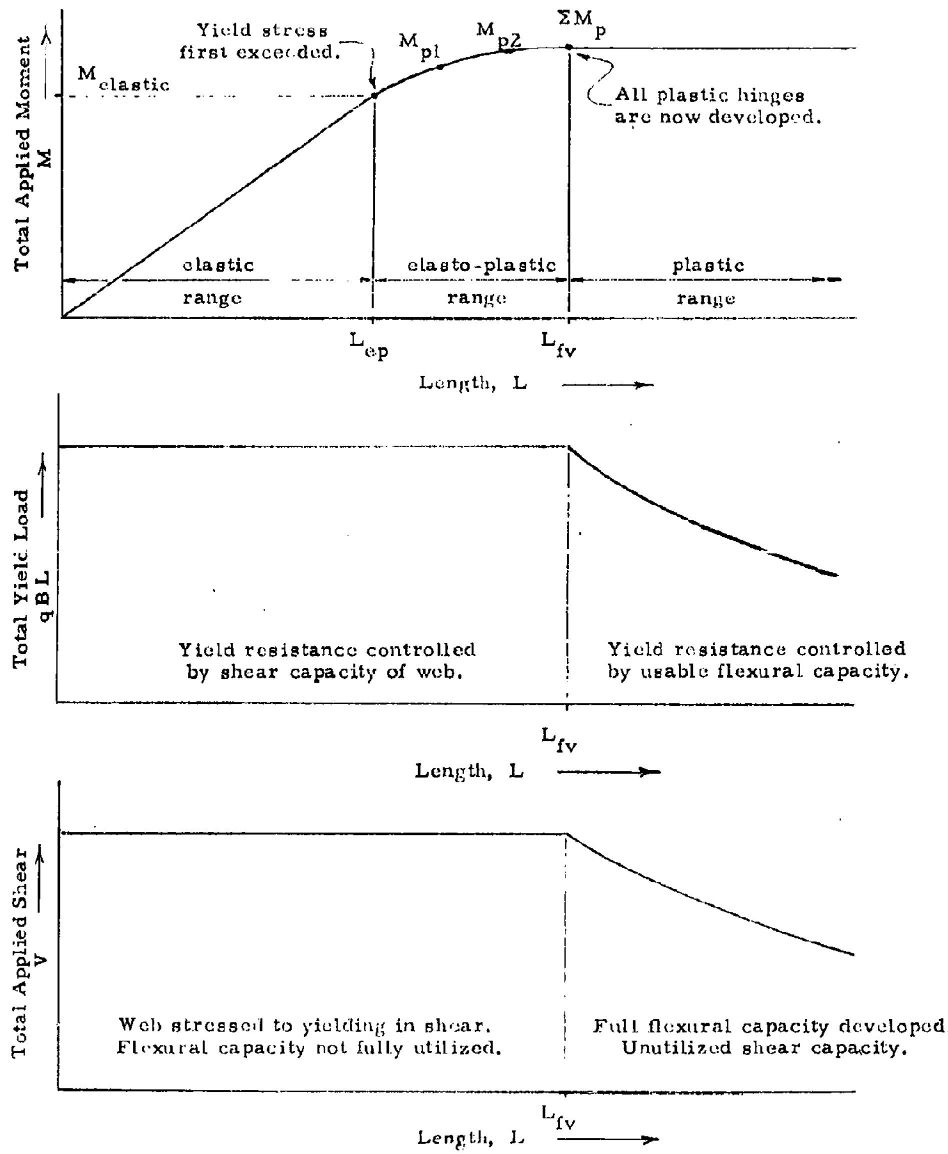


Figure 3-2

IDEALIZED RELATIONSHIPS BETWEEN YIELD LOAD, FAILURE MODE, AND BEAM LENGTH FOR STEEL BEAMS

Equating q_f and q_v at $L = L_{fv}$,

$$\frac{8\,Z\,f_{dy}}{144\,L_{fv}^2} = \frac{1.2\,A_w\,f_{dy}}{12\,L_{fv}}$$

$$L_{fv} = \frac{Z}{1.8\,A_w} \qquad \text{for } q_f = q_v \tag{3.23.1}$$

where

B = center-to-center spacing of beams, (ft)

V_p = total shear causing full plastification of A_w, (lb)

v_{dy} = dynamic shear yield stress, taken as 0.60 f_{dy}, (psi)

A_w = net area of web of rolled section, (in.)

= $d_w \times t_w$, where d_w is net depth of web, (in.), and t_w is thickness of web, (in.)

= $A - (2\,bt_f)$ where b is width of flange, (in.), and t_f is thickness of flange, (in.)

Thus, there are two regions of design interest for a simply supported steel beam which is at incipient plastic collapse under an equivalent uniform load. If $L < L_{fv}$, shearing stresses in the web control the design and

$$\frac{q_v\,B\,L}{f_{dy}} = \frac{A_w}{120} \tag{3.23.2}$$

If $L > L_{fv}$, then flexural stresses on the gross section control design and

$$\frac{q_f\,B\,L^2}{f_{dy}} = \frac{Z}{216} \tag{3.23.3}$$

Finally, if $L = L_{fv}$, then $q_v = q_f$ and

$$\frac{qB}{f_{dy}} = \frac{0.015 A_w^2}{Z} \tag{3.23.4}$$

(2) Fixed-End Beams

Here, maximum shear and moment both occur at the supports, hence a reduced moment capacity M_{pr} is substituted for M_p.

$$q_f \times 12B = \frac{8M_p + 8M_{pr}}{144L^2} = \frac{8f_{dy}}{144L^2}(Z + Z_r)$$

$$q_v \times 12B = \frac{2V_p}{12L} = \frac{2v_{dy}A_w}{12L} = \frac{1.2A_w f_{dy}}{12L}$$

Equating q_f and q_v at $L = L_{fv}$,

$$\frac{8f_{dy}}{144L_{fv}^2}(Z + Z_r) = \frac{1.2A_w f_{dy}}{12L_{fv}}$$

$$L_{fv} = \frac{(Z + Z_r)}{1.8A_w} \qquad \text{for } q_f = q_v \tag{3.23.5}$$

There are two regions of design interest in the case of the fixed-end beam. Assuming incipient yielding, shearing stresses in the web will control for $L < L_{fv}$ and the design equation is

$$\frac{q_v BL}{f_{dy}} = \frac{A_w}{120} \tag{3.23.6}$$

If $L > L_{fv}$, then flexural stresses will control the design. The reduced section modulus Z_r is substituted for Z, since maximum flexure and shear occur at the same section. For this situation,

$$\frac{q_f B L^2}{f_{dy}} = \frac{(Z + Z_r)}{216} \tag{3.23.7}$$

Finally, if $L = L_{fv}$, then $q_v = q_f$ and

$$\frac{q\ B}{f_{dy}} = \frac{0.015\, A_w^2}{(Z + Z_r)} \tag{3.23.8}$$

(3) <u>One Fixed End, One End Simply-Supported</u>

The relationship between shear and moment is not symmetrical for this case, hence the length L_{ep} becomes of significance. This length, as related to a beam loaded to its ultimate capacity in shear, is defined as the beam length for which the computed elastic moment M_e is numerically equal to the reduced plastic yield moment M_{pr} of the section. This continues the earlier assumption that the numerical value of the peak elastic moment is approximately equal to that of the reduced plastic moment.

Equating M_{pr} and M_e at $L = L_{ep}$, assuming the beam is at incipient shear failure due to load q_v, yields the expression

$$M = Z_r f_{dy} = \frac{q_v \times 12\, B \times 144\, L_{ep}}{8}$$

The expression for total shear capacity of the section at $L = L_{ep}$ is

$$V_p = \frac{q_v \times 12\, B \times 12\, L_{ep}}{2} + \frac{M}{12\, L_e} = 0.60\, A_w\, f_{dy}$$

Solving these equations yields the following expression for L_{ep}.

$$L_{ep} = \frac{A_w f_{dy}}{150 q_v B} \tag{3.23.9}$$

For values of $L < L_{ep}$, shear in the web governs design and the resistance of the beam is expressed by

$$\frac{q_v B L}{f_{dy}} = \frac{A_w}{150} \tag{3.23.10}$$

For values of $L_{ep} < L < L_{fv}$ shear in the web governs and, since moment is essentially constant in this idealized elasto-plastic range, the following equation is approximately valid.

$$V_p = \frac{q_v \times 12 B \times 12 L}{2} + \frac{Z_r f_{dy}}{12 L} = 0.6 A_w f_{dy} \text{ (approximate)}$$

Solving for q_v yields

$$\frac{q_v B L}{f_{dy}} = \frac{A_w}{120} - \frac{Z_r}{864 L} \tag{3.23.11}$$

For values of $L > L_{fv}$, flexural stresses on the net beam section control and

$$\frac{q_f B L^2}{f_{dy}} = \frac{(2 Z + Z_r)}{432} \tag{3.23.12}$$

(4) Selection of Steel Beam

Tables 3-2, 3-3 and 3-4 supply flexural and shear resistance functions for standard rolled beam sections, arranged in decreasing order of L_{fv} values. These resistance functions can be used directly for preliminary design, if the assumptions used in their derivation are accepted. A typical design problem involves the selection of required section and spacing for a beam, with L and q previously specified. Maximum strength utilization for the beam section is realized if $L = L_{fv}$, since the beam is then simultaneously yielding in two modes. Therefore, the tables are first used to locate any beam sections whose characteristic lengths, L_{fv}, are approximately equal to the specified beam span, B. If a suitable section can be located, its resistance function corresponding to $L = L_{fv}$ may then be used to compute the permissible beam spacing. However, when short spans and heavy loads are involved, it will be found that a reasonable beam spacing can only be obtained by selecting a beam section whose characteristic L_{fv} length is appreciably greater than the specified span L. In such cases, shear will control the design and a portion of the potential flexural capacity of the section must remain unused. Figure 3-3 illustrates the limiting flexural capacities for uniformly loaded steel beams, of length L feet, spaced a distance B feet apart. In those few cases where the shearing stress does not control the design, a section with adequate flexural resistance can be selected directly from the tables.

Local buckling constitutes a third possible mode of failure for rolled steel beams. Safeguards against such an occurence are provided by specifying critical limiting dimensions for the beam, as listed below[2]. Reference can be made to Table 3-1 to obtain dimensional properties of the standard sections.

$$b/t_f < 17$$

$$d/t_w < 70 \text{ (with longitudinal stiffeners)}$$

$$L_{cr} \leq \left(5 - 3.33 \frac{M}{M_p}\right) r_g$$

Table 3-2
RESISTANCE FUNCTIONS FOR UNIFORMLY LOADED SIMPLY-SUPPORTED STEEL BEAMS

	L_{fv} ft.	Shape	$\frac{q_v B L}{f_{dy}}$ ($L < L_{fv}$)	$\frac{q B}{f_{dy}}$ ($L = L_{fv}$)	$\frac{q_f B L^2}{f_{dy}}$ ($L > L_{fv}$)
1.	22.11	36 WF 300	0.2627	0.01188	5.808
2.	21.97	36 WF 280	0.2460	0.01120	5.404
3.	21.21	36 WF 260	0.2349	0.01108	4.983
4.	20.91	36 WF 245	0.2230	0.01066	4.663
5.	20.50	36 WF 230	0.2127	0.01037	4.360
6.	20.40	14 WF 426	0.1973	0.009668	4.025
7.	19.98	33 WF 240	0.2123	0.01063	4.242
8.	19.97	14 WF 398	0.1862	0.009323	3.718
9.	19.61	14 WF 370	0.1741	0.008879	3.413
10.	19.53	33 WF 220	0.1983	0.01015	3.872
11.	19.14	33 WF 200	0.1829	0.009556	3.502
12.	19.02	14 WF 314	0.1488	0.007825	2.831
13.	18.97	30 WF 210	0.1793	0.009449	3.401
14.	18.60	30 WF 190	0.1642	0.008826	3.054
15.	18.53	14 WF 287	0.1378	0.007440	2.554
16.	18.35	14 WF 264	0.1268	0.006908	2.326
17.	18.17	14 WF 246	0.1183	0.006512	2.151
18.	18.12	30 WF 172	0.1515	0.008360	2.746
19.	17.99	14 WF 228	0.1099	0.006109	1.978
20.	17.98	14 WF 237	0.1147	0.006377	2.062
21.	17.86	14 WF 219	0.1058	0.005921	1.889
22.	17.69	14 WF 184	0.08833	0.004994	1.562
23.	17.68	14 WF 202	0.09783	0.005534	1.730
24.	17.59	14 WF 211	0.1031	0.005860	1.813
25.	17.55	14 WF 193	0.09367	0.005337	1.644
26.	17.49	24 WF 160	0.1228	0.007020	2.146
27.	17.25	14 WF 158	0.07683	0.004454	1.325
28.	17.25	14 WF 176	0.08625	0.005001	1.488
29.	17.13	27 WF 177	0.1506	0.008793	2.579
30.	17.12	14 WF 150	0.07308	0.004270	1.251

Table 3-2 (Continued)

	L_{fv} ft.	Shape	$\frac{q_v B L}{f_{dy}}$ $(L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
31.	17.09	27 WF 160	0.1367	0.008000	2.335
32.	17.08	14 WF 167	0.08208	0.004805	1.402
33.	16.93	24 WF 145	0.1138	0.006719	1.926
34.	16.79	27 WF 145	0.1247	0.007428	2.093
35.	16.50	14 WF 142	0.07150	0.004334	1.180
36.	16.30	36 WF 194	0.2179	0.01337	3.552
37.	16.29	14 WF 127	0.06417	0.003939	1.045
38.	16.22	14 WF 119	0.06000	0.003700	0.973
39.	16.18	14 WF 136	0.06942	0.004292	1.123
40.	16.18	36 WF 182	0.2052	0.01268	3.320
41.	16.17	24 WF 130	0.1057	0.006535	1.709
42.	16.09	14 WF 103	0.05208	0.003237	0.8380
43.	16.04	36 WF 170	0.1924	0.01200	3.086
44.	15.97	14 WF 111	0.05683	0.003560	0.907
45.	15.86	14 WF 87	0.04417	0.002785	0.7005
46.	15.77	14 WF 95	0.04892	0.003102	0.7714
47.	15.62	21 WF 142	0.1058	0.006776	1.653
48.	15.61	36 WF 160	0.1848	0.01184	2.886
49.	15.58	21 WF 127	0.09442	0.006059	1.471
50.	15.56	33 WF 152	0.1661	0.01068	2.584
51.	15.21	21 WF 112	0.08467	0.005567	1.288
52.	15.18	36 WF 150	0.1768	0.01165	2.684
53.	15.01	33 WF 141	0.1583	0.01054	2.375
54.	14.99	24 WF 120	0.1040	0.006939	1.559
55.	14.98	12 WF 190	0.09633	0.006433	1.443
56.	14.93	24 WF 110	0.09542	0.006393	1.424
57.	14.70	24 WF 100	0.08758	0.005957	1.288
58.	14.59	12 WF 161	0.08225	0.005637	1.200
59.	14.22	33 WF 130	0.1518	0.01067	2.158
60.	14.20	14 WF 84	0.04742	0.003339	0.6734
61.	14.15	12 WF 133	0.06867	0.004855	0.9714

Table 3-2 (Continued)

	L_{fv} ft.	Shape	$\frac{q_v B L}{f_{dy}}$ $(L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
62.	14.03	18 WF 114	0.08183	0.005833	1.148
63.	14.00	12 WF 120	0.06450	0.004818	0.8636
64.	13.94	30 WF 132	0.1450	0.01040	2.022
65.	13.92	36 WF 135	0.1693	0.01216	2.357
66.	13.80	14 WF 78	0.04500	0.003260	0.6212
67.	13.79	14 WF 320	0.1988	0.01442	2.742
68.	13.77	18 WF 105	0.07617	0.005532	1.049
69.	13.66	30 WF 124	0.1380	0.01010	0.886
70.	13.56	18 WF 96	0.07042	0.005195	0.9545
71.	13.42	12 WF 106	0.05633	0.004199	0.7559
72.	13.31	12 WF 99	0.05275	0.003964	0.7020
73.	13.28	12 WF 85	0.04500	0.003389	0.5976
74.	13.26	16 WF 96	0.06500	0.004902	0.8620
75.	13.23	33 WF 118	0.1449	0.01095	1.918
76.	13.15	27 WF 114	0.1208	0.009185	1.588
77.	13.14	30 WF 116	0.1330	0.01012	1.747
78.	13.13	12 WF 92	0.04950	0.003771	0.6498
79.	12.92	12 WF 79	0.04275	0.003310	0.5522
80.	12.85	27 WF 102	0.1097	0.008536	1.409
81.	12.81	12 WF 72	0.03908	0.003052	0.5005
82.	12.80	16 WF 88	0.06117	0.004780	0.7828
83.	12.68	12 WF 65	0.03542	0.002793	0.4492
84.	12.40	27 WF 94	0.1038	0.008370	1.286
85.	12.37	30 WF 108	0.1293	0.01045	1.599
86.	12.27	12 WF 58	0.03267	0.002664	0.4007
87.	12.27	14 WF 74	0.04733	0.003858	0.5808
88.	12.22	10 WF 112	0.05592	0.004575	0.6835
89.	12.09	14 WF 68	0.04400	0.003640	0.5320
90.	12.09	24 WF 94	0.09692	0.008017	1.172
91.	11.94	14 WF 61	0.03975	0.003329	0.4747
92.	11.88	10 WF 100	0.05075	0.004274	0.6027

Table 3-2 (Continued)

	L_{fv} ft.	Shape	$\frac{q_v B L}{f_{dy}}$ $(L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
93.	11.74	24 WF 84	0.08833	0.007525	1.037
94.	11.74	30 WF 99	0.1231	0.01049	1.444
95.	11.66	10 WF 89	0.04550	0.003904	0.5303
96.	11.55	12 WF 53	0.03133	0.002713	0.3620
97.	11.47	27 WF 84	0.09808	0.008555	1.125
98.	11.44	10 WF 77	0.03958	0.003460	0.4529
99.	11.38	10 WF 54	0.02725	0.002395	0.3101
100.	11.36	18 WF 77	0.06533	0.005750	0.7424
101.	11.36	18 WF 85	0.07233	0.006369	0.8215
102.	11.34	10 WF 66	0.03383	0.002983	0.3838
103.	11.34	21 WF 96	0.09233	0.008142	1.0470
104.	11.33	10 WF 60	0.03075	0.002714	0.3485
105.	11.20	24 WF 76	0.08267	0.007381	0.9259
106.	11.14	24 I 105.9	0.1135	0.01019	1.2640
107.	11.12	18 WF 70	0.06025	0.005418	0.6700
108.	11.10	10 WF 72	0.03775	0.003400	0.4192
109.	11.09	10 WF 49	0.02517	0.002269	0.2791
110.	11.07	21 WF 82	0.08017	0.007244	0.8872
111.	11.00	18 WF 64	0.05542	0.005040	0.6094
112.	10.73	18 WF 45	0.03875	0.003611	0.4158
113.	10.63	21 WF 73	0.07492	0.007049	0.7963
114.	10.48	16 WF 78	0.06425	0.006131	0.6734
115.	10.43	21 WF 68	0.07083	0.006790	0.7391
116.	10.38	14 WF 53	0.03892	0.003749	0.4040
117.	10.38	24 WF 68	0.07817	0.007531	0.8114
118.	10.33	16 WF 71	0.05900	0.005712	0.6094
119.	10.20	14 WF 48	0.03567	0.003499	0.3636
120.	10.15	16 WF 64	0.05375	0.005297	0.5455
121.	10.13	21 WF 62	0.06583	0.006502	0.6667
122.	10.13	24 I 79.9	0.09275	0.009158	0.9394
123.	9.98	12 WF 50	0.03375	0.003383	0.3367

Table 3-2 (Continued)

	L_{fv} ft.	Shape	$\frac{q_v B L}{f_{dy}}$ $(L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
124.	9.97	12 WF 40	0.02675	0.002684	0.2667
125.	9.95	14 WF 43	0.03242	0.003260	0.3224
126.	9.95	16 WF 58	0.04942	0.004968	0.4916
127.	9.85	12 WF 45	0.03050	0.003098	0.3003
128.	9.82	10 WF 45	0.02592	0.002641	0.2544
129.	9.71	18 WF 60	0.05842	0.006016	0.5673
130.	0.52	24 I 120	0.1450	0.01523	1.3800
131.	9.50	8 WF 67	0.03417	0.003597	0.3246
132.	9.43	18 WF 55	0.05483	0.005818	0.5168
133.	9.42	8 WF 48	0.02408	0.002556	0.2269
134.	9.41	21 WF 55	0.06175	0.006566	0.5808
135.	9.26	18 WF 50	0.05033	0.005433	0.4663
136.	9.25	10 WF 39	0.02350	0.002541	0.2173
137.	9.14	8 WF 58	0.03033	0.003319	0.2773
138.	9.04	16 WF 50	0.04750	0.005256	0.4293
139.	8.81	24 I 90	0.1158	0.01313	1.0200
140.	8.80	16 WF 45	0.04325	0.004917	0.3805
141.	8.75	16 WF 40	0.03850	0.004403	0.3367
142.	8.57	8 WF 35	0.01875	0.002189	0.1606
143.	8.52	8 WF 40	0.02167	0.002542	0.1847
144.	8.40	12 WF 36	0.02833	0.003373	0.2380
145.	8.33	14 WF 38	0.03417	0.004101	0.2847
146.	8.32	10 WF 33	0.02158	0.002594	0.1796
147.	8.31	20 I 85	0.09883	0.01189	0.8215
148.	8.29	20 I 65.4	0.07675	0.009258	0.6364
149.	8.25	12 WF 31	0.02467	0.002990	0.2035
150.	8.20	8 WF 31	0.01717	0.002094	0.1407
151.	8.05	14 WF 34	0.03133	0.003891	0.2524
152.	7.98	24 I 100	0.1386	0.01737	1.1060
153.	7.92	16 WF 36	0.03733	0.004713	0.2958
154.	7.87	12 WF 27	0.02233	0.002838	0.1758

Table 3-2 (Continued)

	L_{fv} ft.	Shape		$\frac{q_v B L}{f_{dy}}$ ($L < L_{fv}$)	$\frac{q B}{f_{dy}}$ ($L = L_{fv}$)	$\frac{q_f B L^2}{f_{dy}}$ ($L > L_{fv}$)
155.	7.54	18 I	54.7	0.06367	0.008449	0.4798
156.	7.48	8 M	24	0.01450	0.001939	0.1084
157.	7.41	8 WF	28	0.01692	0.002282	0.1254
158.	7.39	14 WF	30	0.02950	0.003992	0.2180
159.	7.34	20 I	95	0.1211	0.01650	0.8889
160.	7.33	8 WF	24	0.01458	0.001990	0.1069
161.	7.28	16 B	31	0.03425	0.004702	0.2495
162.	7.25	10 WF	29	0.02217	0.003060	0.1606
163.	7.13	20 I	75	0.09842	0.01380	0.7020
164.	7.06	10 WF	25	0.01933	0.002738	0.1365
165.	6.82	8 M	34.3	0.02225	0.003260	0.1519
166.	6.76	15 I	42.9	0.04700	0.006958	0.3175
167.	6.64	14 B	26	0.02775	0.004178	0.1843
168.	6.50	16 B	26	0.03117	0.004793	0.2027
169.	6.32	6 M	25	0.01308	0.002071	0.0827
170.	6.17	6 M	20	0.01092	0.001770	0.0673
171.	6.09	14 B	22	0.02500	0.004107	0.1522
172.	6.06	10 WF	21	0.01842	0.003039	0.1116
173.	6.04	6 WF	25	0.01458	0.002416	0.0880
174.	6.04	12 I	31.8	0.03183	0.005267	0.1924
175.	5.93	12 I	40.8	0.04092	0.006897	0.2428
176.	5.81	18 I	70	0.09850	0.01695	0.5724
177.	5.80	8 WF	20	0.01525	0.002632	0.0884
178.	5.60	15 I	50	0.06308	0.01126	0.3535
179.	5.56	10 I	25.4	0.02333	0.004195	0.1298
180.	5.47	12 B	22	0.02483	0.004540	0.1359
181.	5.28	12 I	35	0.03892	0.007375	0.2054
182.	5.05	10 B	19	0.01975	0.003908	0.0998
183.	5.00	12 B	19	0.02292	0.004581	0.1146
184.	4.79	14 B	17.2	0.02358	0.004924	0.1130
185.	4.59	12 I	50	0.06117	0.01332	0.2808

Table 3-3

RESISTANCE FUNCTIONS FOR UNIFORMLY-LOADED, FIXED-END STEEL BEAMS

	L_{fv} ft.	Shape	$\frac{q_v B L}{f_{dy}}$ $(L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
1.	39.59	36 WF 300	0.2627	0.006636	10.398
2.	39.30	36 WF 280	0.2460	0.006260	9.668
3.	39.05	14 WF 426	0.1973	0.005051	7.703
4.	38.18	14 WF 398	0.1862	0.004876	7.109
5.	37.79	36 WF 260	0.2349	0.006217	8.877
6.	37.46	14 WF 370	0.1741	0.004647	6.522
7.	37.19	36 WF 245	0.2230	0.005998	8.292
8.	36.37	36 WF 230	0.2127	0.005848	7.734
9.	36.29	14 WF 314	0.1488	0.004102	5.401
10.	35.69	33 WF 240	0.2123	0.005949	7.579
11.	35.30	14 WF 287	0.1378	0.003905	4.866
12.	34.95	14 WF 264	0.1268	0.003627	4.429
13.	34.80	33 WF 220	0.1983	0.005698	6.898
14.	34.59	14 WF 246	0.1183	0.003421	4.093
15.	34.23	14 WF 228	0.1099	0.003211	3.763
16.	34.21	14 WF 237	0.1147	0.003352	3.923
17.	34.08	30 WF 210	0.1793	0.005259	6.110
18.	34.02	33 WF 200	0.1829	0.005377	6.223
19.	33.97	14 WF 219	0.1058	0.003113	3.592
20.	33.62	14 WF 184	0.08833	0.002627	2.970
21.	33.60	14 WF 202	0.09783	0.002912	3.288
22.	33.43	14 WF 211	0.1031	0.003084	3.446
23.	33.35	30 WF 190	0.1642	0.004923	5.475
24.	33.35	14 WF 193	0.09367	0.002809	3.123
25.	32.75	14 WF 153	0.07683	0.002347	2.516
26.	32.74	14 WF 176	0.08625	0.002635	2.824
27.	32.48	14 WF 150	0.07308	0.002250	2.374
28.	32.41	14 WF 167	0.08208	0.002533	2.661
29.	32.39	30 WF 170	0.1515	0.004678	4.907
30.	31.85	24 WF 160	0.1228	0.003854	3.910
31.	31.24	14 WF 142	0.07150	0.002289	2.234

Table 3-3 (Continued)

	L_{fv} ft.	Shape	$\frac{q_v B L}{f_{dy}}$ $(L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
32.	30.83	14 WF 127	0.06417	0.002081	1.978
33.	30.79	27 WF 177	0.1506	0.004891	4.637
34.	30.74	24 WF 145	0.1138	0.003701	3.497
35.	30.71	27 WF 160	0.1367	0.004451	4.197
36.	30.67	14 WF 119	0.06000	0.001956	1.841
37.	30.61	14 WF 136	0.06942	0.002269	2.124
38.	30.43	14 WF 103	0.05208	0.001712	1.585
39.	30.18	14 WF 111	0.05683	0.001884	1.715
40.	30.10	27 WF 145	0.1247	0.004141	3.753
41.	29.97	14 WF 87	0.04417	0.001474	1.324
42.	29.78	14 WF 95	0.04892	0.001643	1.457
43.	29.22	24 WF 130	0.1057	0.003616	3.088
44.	28.57	21 WF 142	0.1058	0.003706	3.023
45.	28.49	21 WF 127	0.09442	0.003314	2.690
46.	28.43	12 WF 190	0.09633	0.003388	2.740
47.	27.88	36 WF 194	0.2179	0.007817	6.076
48.	27.74	21 WF 112	0.08467	0.003052	2.349
49.	27.67	12 WF 161	0.08225	0.002973	2.276
50.	27.64	36 WF 182	0.2052	0.007423	5.671
51.	27.36	36 WF 170	0.1924	0.007035	5.264
52.	26.86	24 WF 120	0.1040	0.003872	2.793
53.	26.77	12 WF 133	0.06867	0.002565	1.839
54.	26.76	33 WF 152	0.1661	0.006208	4.444
55.	26.73	24 WF 110	0.09542	0.003570	2.551
56.	26.66	14 WF 84	0.04742	0.001779	1.264
57.	26.50	36 WF 160	0.1848	0.006975	4.898
58.	26.30	24 WF 100	0.08758	0.003332	2.302
59.	25.86	14 WF 78	0.04500	0.001741	1.164
60.	25.82	14 WF 320	0.1988	0.007700	5.135
61.	25.76	18 WF 114	0.08183	0.003176	2.109
62.	25.66	33 WF 141	0.1583	0.006168	4.060

Table 3-3 (Continued)

	L_{fv} ft.	Shape	$\frac{q_v B L}{f_{dy}}$ $(L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
63.	25.63	36 WF 150	0.1768	0.006900	4.533
64.	25.31	12 WF 106	0.05633	0.002225	1.426
65.	25.27	12 WF 120	0.06450	0.002553	1.630
66.	25.24	18 WF 105	0.07617	0.003017	1.923
67.	25.14	36 WF 135	0.1693	0.007318	3.915
68.	25.08	12 WF 99	0.05275	0.002102	1.324
69.	25.05	12 WF 85	0.04500	0.001797	1.127
70.	24.82	18 WF 96	0.07042	0.002838	1.748
71.	24.73	12 WF 92	0.04950	0.002001	1.225
72.	24.50	16 WF 96	0.06500	0.002645	1.592
73.	24.32	12 WF 79	0.04275	0.001758	1.040
74.	24.10	12 WF 72	0.03908	0.001622	0.9418
75.	24.08	33 WF 130	0.1518	0.006302	3.654
76.	23.95	30 WF 132	0.1450	0.006053	3.474
77.	23.85	12 WF 65	0.03542	0.001485	0.8447
78.	23.56	16 WF 88	0.06117	0.002595	1.442
79.	23.39	30 WF 124	0.1380	0.005900	3.228
80.	23.21	10 WF 112	0.05592	0.002409	1.298
81.	23.01	12 WF 58	0.03267	0.001420	0.7518
82.	22.79	14 WF 74	0.04733	0.002077	1.079
83.	22.77	27 WF 114	0.1208	0.005306	2.748
84.	22.52	10 WF 100	0.05075	0.002254	1.143
85.	22.43	14 WF 68	0.04400	0.001962	0.9867
86.	22.34	30 WF 116	0.1330	0.005953	2.972
87.	22.17	27 WF 102	0.1097	0.004948	2.431
88.	22.13	14 WF 61	0.03975	0.001796	0.8798
89.	22.10	33 WF 118	0.1449	0.006558	3.203
90.	22.08	10 WF 89	0.04550	0.002061	1.004
91.	21.65	10 WF 77	0.03958	0.001829	0.8569
92.	21.59	12 WF 53	0.03133	0.001452	0.6764
93.	21.52	10 WF 54	0.02725	0.001266	0.5865

Table 3-3 (Continued)

	L_{fv} ft.	Shape	$\frac{q_v B L}{f_{dy}}$ $(L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
94.	21.46	10 WF 66	0.03383	0.001577	0.7259
95.	21.43	10 WF 60	0.03075	0.001435	0.6590
96.	21.26	27 WF 94	0.1038	0.004880	2.206
97.	21.05	24 WF 94	0.09692	0.004605	2.040
98.	20.97	10 WF 72	0.03775	0.001800	0.7918
99.	20.95	10 WF 49	0.02517	0.001202	0.5272
100.	20.81	30 WF 108	0.1293	0.006210	2.690
101.	20.43	18 WF 77	0.06533	0.003198	1.335
102.	20.41	18 WF 85	0.07233	0.003542	1.477
103.	20.35	24 WF 84	0.08833	0.004342	1.797
104.	20.00	21 WF 96	0.09233	0.004616	1.847
105.	19.95	18 WF 70	0.06025	0.003021	1.202
106.	19.70	18 WF 64	0.05542	0.002813	1.092
107.	19.53	18 WF 45	0.03875	0.001984	0.7569
108.	19.53	30 WF 99	0.1231	0.006300	2.405
109.	19.45	21 WF 82	0.08017	0.004121	1.560
110.	19.40	27 WF 84	0.09808	0.005057	1.903
111.	19.27	24 WF 76	0.08267	0.004291	1.593
112.	19.25	24 I 105.9	0.1135	0.005897	2.185
113.	19.01	14 WF 53	0.03892	0.002047	0.7398
114.	18.94	16 WF 78	0.06425	0.003393	1.217
115.	18.64	14 WF 48	0.03567	0.001914	0.6647
116.	18.63	16 WF 71	0.05900	0.003167	1.099
117.	18.52	21 WF 73	0.07492	0.004047	1.387
118.	18.44	12 WF 50	0.03375	0.001831	0.6222
119.	18.42	12 WF 40	0.02675	0.001452	0.4927
120.	18.40	10 WF 45	0.02592	0.001409	0.4767
121.	18.27	16 WF 64	0.05375	0.002942	0.9821
122.	18.18	12 WF 45	0.03050	0.001678	0.5545
123.	18.13	14 WF 43	0.03242	0.001788	0.5879
124.	18.11	21 WF 68	0.07083	0.003910	1.284

Table 3-3 (Continued)

	L_{fv} ft.	Shape	$\frac{q_v B L}{f_{dy}}$ $(L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
125.	18.01	8 WF 67	0.03417	0.001897	0.6153
126.	17.87	16 WF 58	0.04942	0.002766	0.8831
127.	17.85	8 WF 48	0.02408	0.001349	0.4300
128.	17.62	24 WF 68	0.07817	0.004435	1.378
129.	17.52	21 WF 62	0.06583	0.003761	1.153
130.	17.29	8 WF 58	0.03033	0.001755	0.5245
131.	17.26	10 WF 39	0.02350	0.001361	0.4057
132.	17.17	24 I 79.9	0.09275	0.005405	0.1592
133.	17.08	18 WF 60	0.05842	0.003420	0.9979
134.	16.51	18 WF 55	0.05483	0.003322	0.9051
135.	16.18	18 WF 50	0.05033	0.003110	0.8146
136.	16.14	8 WF 35	0.01875	0.001162	0.3026
137.	16.07	21 WF 55	0.06175	0.003844	0.9920
138.	16.06	8 WF 40	0.02167	0.001349	0.3479
139.	16.01	24 I 120	0.1450	0.009058	2.322
140.	15.99	16 WF 50	0.04750	0.002971	0.7596
141.	15.51	16 WF 45	0.04325	0.002789	0.6708
142.	15.41	10 WF 33	0.02158	0.001401	0.3327
143.	15.40	8 WF 31	0.01717	0.001115	0.2644
144.	15.40	16 WF 40	0.03850	0.002500	0.5929
145.	15.25	12 WF 36	0.02833	0.001858	0.4322
146.	14.95	12 WF 31	0.02467	0.001650	0.3688
147.	14.84	14 WF 38	0.03417	0.002302	0.5072
148.	14.53	24 I 90	0.1158	0.007965	1.682
149.	14.29	14 WF 34	0.03133	0.002193	0.4477
150.	14.19	12 WF 27	0.02233	0.001574	0.3169
151.	14.10	20 I 85	0.09883	0.007010	1.394
152.	14.02	20 I 65.4	0.07675	0.005474	1.076
153.	13.95	8 M 24	0.01450	0.001040	0.2022
154.	13.84	8 WF 28	0.01692	0.001223	0.2341
155.	13.76	16 WF 36	0.03733	0.002713	0.5138

Table 3-3 (Continued)

	L_{fv} ft.	Shape		$\frac{q_v B L}{f_{dy}}$ $(L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
156.	13.67	8 WF	24	0.01458	0.001067	0.1993
157.	13.21	10 WF	29	0.02217	0.001678	0.2929
158.	12.96	14 WF	30	0.02950	0.002277	0.3823
159.	12.86	24 I	100	0.1386	0.01077	1.783
160.	12.84	10 WF	25	0.01933	0.001505	0.2483
161.	12.76	18 I	54.7	0.06367	0.004989	0.8126
162.	12.66	8 M	34.3	0.02225	0.001758	0.2817
163.	12.49	16 B	31	0.03425	0.002742	0.4278
164.	12.15	20 I	95	0.1211	0.009960	1.472
165.	11.94	6 M	25	0.01308	0.001096	0.1562
166.	11.70	20 I	75	0.09842	0.008409	1.152
167.	11.61	6 M	20	0.01092	0.0009405	0.1267
168.	11.60	15 I	42.9	0.04700	0.004052	0.5452
169.	11.47	14 B	26	0.02775	0.002419	0.3183
170.	11.31	6 WF	25	0.01458	0.001289	0.1650
171.	11.10	6 WF	20	0.01175	0.001058	0.1305
172.	10.93	16 B	26	0.03117	0.002852	0.3406
173.	10.84	10 WF	21	0.01842	0.001699	0.1997
174.	10.57	8 WF	20	0.01525	0.001443	0.1611
175.	10.57	12 I	31.8	0.03183	0.003011	0.3366
176.	10.38	12 I	40.8	0.04092	0.003941	0.4248
177.	10.36	14 B	22	0.02500	0.002413	0.2591
178.	9.87	10 I	25.4	0.02333	0.002364	0.2303
179.	9.35	12 B	22	0.02483	0.002657	0.2322
180.	9.31	18 I	70	0.09850	0.01058	0.9172
181.	9.30	15 I	50	0.06308	0.006787	0.5864
182.	9.29	8 WF	17	0.01417	0.001525	0.1316
183.	9.04	12 I	35	0.03892	0.004306	0.3518
184.	8.98	5 M	18.9	0.01075	0.001197	0.09634
185.	8.84	8 M	17	0.01475	0.001669	0.1303
186.	8.83	6 WF	15.5	0.01092	0.001236	0.09643

Table 3-3 (Continued)

	L_{fv} ft.	Shape		$\frac{q_v B L}{f_{dy}}$ $(L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
187.	8.79	10 B	19	0.01975	0.002247	0.1736
188.	8.43	8 I	184	0.01608	0.001909	0.1355
189.	8.41	12 B	19	0.02292	0.002724	0.1928
190.	8.39	6 B	16	0.01175	0.001401	0.09855
191.	7.80	10 B	17	0.01892	0.002426	0.1475
192.	7.71	14 B	17.2	0.02358	0.003060	0.1818
193.	7.70	12 I	50	0.06117	0.007948	0.4708
194.	7.69	7 I	15.3	0.01292	0.001680	0.09931
195.	7.17	8 B	15	0.01533	0.002139	0.1099
196.	7.08	12 B	16.5	0.02200	0.003107	0.1558
197.	6.95	6 I	12.5	0.01008	0.001451	0.07007
198.	6.86	12 B	14	0.01908	0.002782	0.1309
199.	6.82	10 B	15	0.01817	0.002664	0.1239
200.	6.76	8 B	10	0.01058	0.001567	0.07150
201.	6.60	10 B	11.5	0.01417	0.002146	0.09352
202.	6.49	8 M	20	0.02150	0.003312	0.1396
203.	6.29	8 B	13	0.01433	0.002279	0.09015
204.	6.26	12 Jr	11.8	0.01675	0.002675	0.1049
205.	6.04	10 I	35	0.04467	0.007402	0.2696
206.	5.77	8 I	23	0.02625	0.004552	0.1514
207.	5.55	10 Jr	9	0.01242	0.002237	0.06893
208.	4.84	7 I	20	0.02333	0.004826	0.1128

Table 3-4
RESISTANCE FUNCTIONS FOR UNIFORMLY-LOADED STEEL BEAMS, ONE END FIXED AND ONE END SIMPLY-SUPPORTED

	L_{fv} ft.	Shape	L_{ep} ft.	$\frac{q_v B L}{f_{dy}}$ $(L < L_{ep})$	$\frac{q_v B L}{f_{dy}}$ $(L_{ep} < L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
1.	35.22	36 WF 300	21.85	0.2101	0.2627 - 1.148/L	0.006533	8.103
2.	34.97	36 WF 280	21.66	0.1968	0.2460 - 1.066/L	0.006164	7.536
3.	34.39	14 WF 426	23.31	0.1578	0.1973 - 0.9196/L	0.004958	5.864
4.	33.64	36 WF 260	20.72	0.1879	0.2349 - 0.9735/L	0.006122	6.930
5.	33.63	14 WF 398	22.77	0.1489	0.1862 - 0.8478/L	0.004786	5.413
6.	33.12	36 WF 245	20.34	0.1784	0.2230 - 0.9073/L	0.005906	6.478
7.	33.00	14 WF 370	22.32	0.1393	0.1741 - 0.7770/L	0.004562	4.967
8.	32.40	36 WF 230	19.83	0.1701	0.2127 - 0.8436/L	0.005760	6.047
9.	31.97	14 WF 314	21.58	0.1191	0.1488 - 0.6425/L	0.004027	4.116
10.	31.77	33 WF 240	19.64	0.1699	0.2123 - 0.8341/L	0.005858	5.911
11.	31.11	14 WF 287	20.97	0.1103	0.1378 - 0.5780/L	0.003834	3.710
12.	30.98	33 WF 220	19.08	0.1586	0.1983 - 0.7566/L	0.005611	5.386
13.	30.80	14 WF 264	20.74	0.1014	0.1268 - 0.5259/L	0.003561	3.378
14.	30.49	14 WF 246	20.52	0.09467	0.1183 - 0.4857/L	0.003359	3.122
15.	30.30	30 WF 210	18.89	0.1434	0.1793 - 0.6773/L	0.005177	4.755
16.	30.29	33 WF 200	18.60	0.1463	0.1829 - 0.6803/L	0.005296	4.862
17.	30.17	14 WF 228	20.30	0.08793	0.1099 - 0.4462/L	0.003153	2.870
18.	30.16	14 WF 237	20.29	0.09173	0.1147 - 0.4652/L	0.003291	2.993
19.	29.94	14 WF 219	20.13	0.08460	0.1058 - 0.4258/L	0.003057	2.741
20.	29.66	30 WF 190	18.43	0.1313	0.1642 - 0.6052/L	0.004847	4.264
21.	29.64	14 WF 184	19.92	0.07067	0.08833 - 0.3519/L	0.002580	2.266
22.	29.62	14 WF 202	19.91	0.07827	0.09783 - 0.3895/L	0.002859	2.509
23.	29.47	14 WF 211	19.80	0.08247	0.1031 - 0.4082/L	0.003028	2.630
24.	29.40	14 WF 193	19.74	0.07493	0.09367 - 0.3699/L	0.002758	2.384
25.	28.87	14 WF 158	19.37	0.06147	0.07683 - 0.2976/L	0.002304	1.921
26.	28.87	14 WF 176	19.37	0.06900	0.08625 - 0.3341/L	0.002587	2.156
27.	28.82	30 WF 172	17.83	0.1212	0.1515 - 0.5403/L	0.004606	3.826
28.	28.64	14 WF 150	19.20	0.05847	0.07308 - 0.2807/L	0.002210	1.812
29.	28.58	14 WF 167	19.16	0.06567	0.08208 - 0.3146/L	0.002487	2.031
30.	28.26	24 WF 160	17.96	0.09820	0.1228 - 0.4408/L	0.003792	3.028
31.	27.56	14 WF 142	18.43	0.05720	0.07150 - 0.2636/L	0.002248	1.707
32.	27.38	27 WF 177	17.08	0.1205	0.1506 - 0.5144/L	0.004814	3.608
33.	27.31	27 WF 160	17.03	0.1093	0.1367 - 0.4654/L	0.004381	3.266
34.	27.29	24 WF 145	17.26	0.09100	0.1138 - 0.3927/L	0.003641	2.711
35.	27.19	14 WF 127	18.17	0.05133	0.06417 - 0.2332/L	0.002044	1.512
36.	27.06	14 WF 119	18.08	0.04800	0.06000 - 0.2169/L	0.001921	1.407
37.	27.00	14 WF 136	18.03	0.05553	0.06942 - 0.2503/L	0.002228	1.623

Table 3-4 (Continued)
RESISTANCE FUNCTIONS FOR UNIFORMLY-LOADED STEEL BEAMS, ONE END FIXED AND ONE END SIMPLY-SUPPORTED

	L_{fv} ft.	Shape	L_{ep} ft.	$\frac{q_v B L}{f_{dy}}$ $(L < L_{ep})$	$\frac{q_v B L}{f_{dy}}$ $(L_{ep} < L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
38.	26.84	14 WF 103	17.92	0.04167	0.05208 - 0.1867/L	0.001681	1.211
39.	26.78	27 WF 145	16.65	0.09973	0.1247 - 0.4151/L	0.004077	2.923
40.	26.63	14 WF 111	17.76	0.04547	0.05683 - 0.2019/L	0.001850	1.311
41.	26.44	14 WF 87	17.63	0.03533	0.04417 - 0.1558/L	0.001448	1.012
42.	26.28	14 WF 95	17.52	0.03913	0.04892 - 0.1714/L	0.001613	1.114
43.	25.96	24 WF 130	16.31	0.08453	0.1057 - 0.3448/L	0.003559	2.398
44.	25.33	21 WF 142	16.18	0.08467	0.1058 - 0.3424/L	0.003645	2.338
45.	25.26	21 WF 127	16.13	0.07553	0.09442 - 0.3046/L	0.003260	2.081
46.	25.07	12 WF 190	16.83	0.07707	0.09633 - 0.3242/L	0.003326	2.091
47.	24.99	36 WF 194	14.48	0.1743	0.2179 - 0.6309/L	0.007711	4.814
48.	24.78	36 WF 182	14.33	0.1641	0.2052 - 0.5879/L	0.007323	4.496
49.	24.53	36 WF 170	14.15	0.1539	0.1924 - 0.5444/L	0.006940	4.175
50.	24.40	12 WF 161	16.35	0.06580	0.08225 - 0.2689/L	0.002919	1.738
51.	24.21	21 WF 112	15.66	0.06773	0.08467 - 0.2653/L	0.003002	1.818
52.	23.96	33 WF 152	14.00	0.1329	0.1661 - 0.4649/L	0.006122	3.514
53.	23.89	24 WF 120	14.84	0.08320	0.1040 - 0.3086/L	0.003812	2.176
54.	23.78	24 WF 110	14.76	0.07633	0.09542 - 0.2816/L	0.003514	1.987
55.	23.78	36 WF 160	13.61	0.1479	0.1848 - 0.5032/L	0.006883	3.892
56.	23.62	12 WF 133	15.79	0.05493	0.06867 - 0.2168/L	0.002519	1.405
57.	23.55	14 WF 84	15.56	0.03793	0.04742 - 0.1476/L	0.001748	0.9685
58.	23.39	24 WF 100	14.48	0.07007	0.08758 - 0.2536/L	0.003281	1.795
59.	23.02	36 WF 150	13.07	0.1415	0.1768 - 0.4623/L	0.006810	3.608
60.	23.00	33 WF 141	13.31	0.1266	0.1583 - 0.4213/L	0.006085	3.218
61.	22.91	12 WF 120	14.84	0.05160	0.06450 - 0.1915/L	0.002508	1.247
62.	22.84	14 WF 78	15.06	0.03600	0.04500 - 0.1356/L	0.001710	0.8924
63.	22.82	18 WF 114	14.67	0.06547	0.08183 - 0.2401/L	0.003123	1.628
64.	22.82	14 WF 320	15.04	0.1591	0.1988 - 0.5982/L	0.007566	3.938
65.	22.37	18 WF 105	14.34	0.06093	0.07617 - 0.2185/L	0.002967	1.486
66.	22.34	12 WF 106	14.88	0.04507	0.05633 - 0.1676/L	0.002185	1.091
67.	22.14	12 WF 99	14.74	0.04220	0.05275 - 0.1555/L	0.002064	1.013
68.	22.10	12 WF 85	14.71	0.03600	0.04500 - 0.1324/L	0.001765	0.8623
69.	22.01	18 WF 96	14.08	0.05633	0.07042 - 0.1983/L	0.002791	1.351
70.	21.83	12 WF 92	14.52	0.03960	0.04950 - 0.1437/L	0.001965	0.9373
71.	21.69	16 WF 96	14.04	0.05200	0.06500 - 0.1826/L	0.002609	1.227
72.	21.62	33 WF 130	12.32	0.1214	0.1518 - 0.3740/L	0.006220	2.906
73.	21.47	12 WF 79	14.25	0.03420	0.04275 - 0.1218/L	0.001727	0.7959
74.	21.45	30 WF 132	12.51	0.1160	0.1450 - 0.3629/L	0.005970	2.748

Table 3-4 (Continued)
RESISTANCE FUNCTIONS FOR UNIFORMLY-LOADED STEEL BEAMS, ONE END FIXED AND ONE END SIMPLY-SUPPORTED

	L_{fv} ft.	Shape	L_{ep} ft.	$\frac{q_v B L}{f_{dy}}$ $(L < L_{ep})$	$\frac{q_v B L}{f_{dy}}$ $(L_{ep} < L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
75.	21.27	12 WF 72	14.11	0.03127	0.03908 - 0.1103/L	0.001593	0.7211
76.	21.06	12 WF 65	13.96	0.02833	0.03542 - 0.09888/L	0.001459	0.6469
77.	20.95	30 WF 124	12.16	0.1104	0.1380 - 0.3356/L	0.005820	2.557
78.	20.87	16 WF 88	13.47	0.04893	0.06117 - 0.1648/L	0.002552	1.112
79.	20.83	36 WF 135	11.51	0.1354	0.1693 - 0.3895/L	0.007228	3.136
80.	20.46	10 WF 112	13.74	0.04473	0.05592 - 0.1536/L	0.002366	0.9907
81.	20.37	27 WF 114	12.02	0.09660	0.1208 - 0.2902/L	0.005231	2.168
82.	20.33	12 WF 58	13.43	0.02613	0.03267 - 0.08777/L	0.001395	0.5762
83	20.16	14 WF 74	13.15	0.03787	0.04733 - 0.1244/L	0.002042	0.8297
84.	20.04	30 WF 116	11.51	0.1064	0.1330 - 0.3061/L	0.005874	2.360
85.	19.88	33 WF 118	11.09	0.1159	0.1449 - 0.3213/L	0.006476	2.560
86.	19.86	10 WF 100	13.30	0.04060	0.05075 - 0.1350/L	0.002214	0.8727
87.	19.84	14 WF 68	12.92	0.03520	0.04400 - 0.1137/L	0.001929	0.7594
88.	19.84	27 WF 102	11.65	0.08773	0.1097 - 0.2550/L	0.004879	1.920
89.	19.59	14 WF 61	12.74	0.03180	0.03975 - 0.1013/L	0.001766	0.6773
90.	19.47	10 WF 89	13.03	0.03640	0.04555 - 0.1185/L	0.002024	0.7674
91.	19.10	10 WF 77	12.76	0.03167	0.03958 - 0.1010/L	0.001796	0.6549
92.	19.08	12 WF 53	12.55	0.02507	0.03133 - 0.07862/L	0.001426	0.5192
93.	19.04	27 WF 94	11.08	0.08300	0.1038 - 0.2300/L	0.004813	1.746
94.	18.99	10 WF 54	12.68	0.02180	0.02725 - 0.06911/L	0.001243	0.4483
95.	18.93	10 WF 66	12.64	0.02707	0.03383 - 0.08552/L	0.001549	0.5549
96.	18.91	10 WF 60	12.62	0.02460	0.03075 - 0.07762/L	0.001409	0.5037
97.	18.81	24 WF 94	11.20	0.07753	0.09692 - 0.2170/L	0.004539	1.606
98.	18.70	30 WF 108	10.55	0.1034	0.1293 - 0.2727/L	0.006131	2.145
99.	18.51	10 WF 72	12.34	0.03020	0.03775 - 0.09315/L	0.001768	0.6055
100.	18.48	10 WF 49	12.32	0.02013	0.02517 - 0.06201/L	0.001180	0.4032
101.	18.20	24 WF 84	10.76	0.07067	0.08833 - 0.1900/L	0.004281	1.417
102.	18.16	18 WF 77	11.34	0.05227	0.06533 - 0.1481/L	0.003148	1.039
103.	18.15	18 WF 85	11.33	0.05787	0.07233 - 0.1639/L	0.003487	1.149
104.	17.84	21 WF 96	10.83	0.07387	0.09233 - 0.2000/L	0.004548	1.447
105.	17.74	18 WF 70	11.03	0.04820	0.06025 - 0.1330/L	0.002974	0.9359
106.	17.59	30 WF 99	9.75	0.09847	0.1231 - 0.2401/L	0.006222	1.925
107.	17.52	18 WF 64	10.88	0.04433	0.05542 - 0.1206/L	0.002770	0.8506
108.	17.42	27 WF 84	9.92	0.07847	0.09808 - 0.1945/L	0.004991	1.514
109.	17.35	21 WF 82	10.48	0.06413	0.08017 - 0.1681/L	0.004061	1.223
110.	17.33	18 WF 45	11.00	0.03100	0.03875 - 0.08527/L	0.001952	0.5864
111.	17.25	24 WF 76	10.08	0.06613	0.08267 - 0.1667/L	0.004232	1.259
112.	17.23	24 I 105.9	10.14	0.09080	0.1135 - 0.2301/L	0.005814	1.725

Table 3-4 (Continued)
RESISTANCE FUNCTIONS FOR UNIFORMLY-LOADED STEEL BEAMS, ONE END FIXED AND ONE END SIMPLY-SUPPORTED

	L_{fv} ft.	Shape	L_{ep} ft.	$\frac{q_v B L}{f_{dy}}$ $(L < L_{ep})$	$\frac{q_v B L}{f_{dy}}$ $(L_{ep} < L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
113.	16.85	14 WF 53	10.78	0.03113	0.03892 - 0.08394/L	0.002014	0.5719
114.	16.83	16 WF 78	10.57	0.05140	0.06425 - 0.1358/L	0.003339	0.9450
115.	16.56	16 WF 71	10.38	0.04720	0.05900 - 0.1225/L	0.003117	0.8544
116.	16.55	21 WF 73	9.85	0.05993	0.07492 - 0.1476/L	0.003989	1.092
117.	16.53	14 WF 48	10.55	0.02853	0.03567 - 0.07526/L	0.001883	0.5142
118.	16.32	12 WF 50	10.57	0.02700	0.03375 - 0.07137/L	0.001800	0.4794
119.	16.31	12 WF 40	10.56	0.02140	0.02675 - 0.05652/L	0.001428	0.3797
120.	16.25	10 WF 45	10.73	0.02073	0.02592 - 0.05559/L	0.001384	0.3656
121.	16.24	16 WF 64	10.16	0.04300	0.05375 - 0.1092/L	0.002896	0.7638
122.	16.19	21 WF 68	9.61	0.05667	0.07083 - 0.1361/L	0.003854	1.011
123.	16.10	12 WF 45	10.42	0.02440	0.03050 - 0.06354/L	0.001650	0.4274
124.	16.09	14 WF 43	10.24	0.02593	0.03242 - 0.06637/L	0.001759	0.4551
125.	15.89	16 WF 58	9.90	0.03953	0.04942 - 0.09788/L	0.002722	0.6873
126.	15.88	8 WF 67	10.64	0.02733	0.03417 - 0.07268/L	0.001863	0.4699
127.	15.81	24 WF 68	9.06	0.06253	0.07817 - 0.1416/L	0.004376	1.095
128.	15.75	8 WF 48	10.54	0.01927	0.02408 - 0.05076/L	0.001325	0.3285
129.	15.67	21 WF 62	9.23	0.05267	0.06583 - 0.1215/L	0.003708	0.9096
130.	15.41	24 I 79.9	8.79	0.07420	0.09275 - 0.1631/L	0.005334	1.266
131.	15.26	10 WF 39	10.02	0.01880	0.02350 - 0.04709/L	0.001338	0.3115
132.	15.25	8 WF 58	10.19	0.02427	0.03033 - 0.06180/L	0.001723	0.4009
133.	15.24	18 WF 60	9.21	0.04673	0.05842 - 0.1076/L	0.003370	0.7826
134.	14.74	18 WF 55	8.85	0.04387	0.05483 - 0.09706/L	0.003274	0.7110
135.	14.45	18 WF 50	8.65	0.04027	0.05033 - 0.09707/L	0.003065	0.6405
136.	14.40	21 WF 55	8.32	0.04940	0.06175 - 0.1028/L	0.003792	0.7864
137.	14.39	24 I 120	8.11	0.1160	0.1450 - 0.2353/L	0.008941	1.851
138.	14.25	8 WF 35	9.47	0.01500	0.01875 - 0.03550/L	0.001141	0.2316
139.	14.25	16 WF 50	8.69	0.03800	0.04750 - 0.08257/L	0.002926	0.5944
140.	14.17	8 WF 40	9.42	0.01733	0.02167 - 0.04081/L	0.001326	0.2663
141.	13.83	16 WF 45	8.39	0.03460	0.04325 - 0.07258/L	0.002748	0.5256
142.	13.74	16 WF 40	8.32	0.03080	0.03850 - 0.06404/L	0.002463	0.4648
143.	13.64	10 WF 33	8.86	0.01727	0.02158 - 0.03826/L	0.001377	0.2561
144.	13.60	8 WF 31	9.01	0.01373	0.01717 - 0.03092/L	0.001095	0.2026
145.	13.54	12 WF 36	8.57	0.02267	0.02833 - 0.04854/L	0.001828	0.3351
146.	13.28	12 WF 31	8.37	0.01973	0.02467 - 0.04131/L	0.001624	0.2862
147.	13.22	14 WF 38	8.14	0.02733	0.03417 - 0.05562/L	0.002267	0.3959
148.	13.10	24 I 90	7.15	0.09260	0.1158 - 0.1655/L	0.007869	1.351
149.	12.73	14 WF 34	7.79	0.02507	0.03133 - 0.04883/L	0.002160	0.3500
150.	12.65	20 I 85	7.24	0.07907	0.09883 - 0.1430/L	0.006918	1.108

Table 3-4 (Continued)
RESISTANCE FUNCTIONS FOR UNIFORMLY-LOADED STEEL BEAMS, ONE END FIXED AND ONE END SIMPLY-SUPPORTED

	L_{fv} ft.	Shape		L_{ep} ft.	$\frac{q_v B L}{f_{dy}}$ $(L < L_{ep})$	$\frac{q_v B L}{f_{dy}}$ $(L_{ep} < L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
151.	12.61	12 WF	27	7.90	0.01787	0.02233 - 0.03527/L	0.001549	0.2463
152.	12.59	20 I	65.4	7.16	0.06140	0.07675 - 0.1100/L	0.005402	0.8563
153.	12.33	8 M	24	8.09	0.01160	0.01450 - 0.02345/L	0.001022	0.1553
154.	12.30	16 WF	36	7.30	0.02987	0.03733 - 0.05451/L	0.002674	0.4048
155.	12.23	8 WF	28	8.03	0.01353	0.01692 - 0.02717/L	0.001201	0.1798
156.	12.08	8 WF	24	7.92	0.01167	0.01458 - 0.02311/L	0.001049	0.1531
157.	11.72	10 WF	29	7.46	0.01773	0.02217 - 0.03306/L	0.001651	0.2267
158.	11.64	24 I	100	6.11	0.1109	0.1386 - 0.1693/L	0.001065	1.445
159.	11.57	14 WF	30	6.96	0.02360	0.02950 - 0.04106/L	0.002244	0.3001
160.	11.46	18 I	54.7	6.53	0.05093	0.06367 - 0.08321/L	0.004923	0.6462
161.	11.40	10 WF	25	7.23	0.01547	0.01933 - 0.02795/L	0.001481	0.1924
162.	11.20	8 M	34.3	7.29	0.01780	0.02225 - 0.03246/L	0.001728	0.2168
163.	11.19	16 B	31	6.51	0.02740	0.03425 - 0.04459/L	0.002705	0.3387
164.	10.95	20 I	95	6.02	0.09687	0.1211 - 0.1458/L	0.009839	1.181
165.	10.56	20 I	75	5.72	0.07873	0.09842 - 0.1125/L	0.008309	0.9270
166.	10.53	6 M	25	7.03	0.01047	0.01308 - 0.01838/L	0.001076	0.1194
167.	10.39	15 I	42.9	6.05	0.03760	0.04700 - 0.05691/L	0.003997	0.4313
168.	10.26	14 B	26	6.03	0.02220	0.02775 - 0.03349/L	0.002386	0.2513
169.	10.00	6 WF	25	6.60	0.01167	0.01458 - 0.01924/L	0.001266	0.1265
170.	9.82	16 B	26	5.53	0.02493	0.03117 - 0.03447/L	0.002816	0.2716
171.	9.65	10 WF	21	5.98	0.01473	0.01842 - 0.02201/L	0.001673	0.1556
172.	9.44	12 I	31.8	5.66	0.02547	0.03183 - 0.03603/L	0.002968	0.2645
173.	9.37	8 WF	20	5.96	0.01220	0.01525 - 0.01819/L	0.001420	0.1248
174.	9.29	14 B	22	5.34	0.02000	0.02500 - 0.02672/L	0.002381	0.2056
175.	9.27	12 I	40.8	5.56	0.03273	0.04092 - 0.04551/L	0.003884	0.3338
176.	8.79	10 I	25.4	5.38	0.01867	0.02333 - 0.02513/L	0.002329	0.1801
177.	8.61	5 WF	16	5.73	0.006867	0.008583- 0.009831/L	0.0008639	0.06411
178.	8.44	18 I	70	4.38	0.07880	0.09850 - 0.08620/L	0.01046	0.7448
179.	8.38	12 B	22	4.85	0.01987	0.02483 - 0.02407/L	0.002621	0.1840
180.	8.37	15 I	50	4.61	0.05047	0.06308 - 0.05821/L	0.006704	0.4700
181.	8.26	8 WF	17	5.16	0.01133	0.01417 - 0.01463/L	0.001501	0.1023
182.	8.10	12 I	35	4.70	0.03113	0.03892 - 0.03659/L	0.004248	0.2786
183.	7.93	5 M	18.9	5.25	0.008600	0.01075 - 0.01130/L	0.001176	0.07394
184.	7.86	8 M	17	4.88	0.01180	0.01475 - 0.01440/L	0.001643	0.1015
185.	7.86	10 B	19	4.67	0.01580	0.01975 - 0.01845/L	0.002215	0.1367
186.	7.82	6 WF	15.5	5.05	0.008733	0.01092 - 0.01102/L	0.001215	0.07439
187.	7.56	12 B	19	4.26	0.01833	0.02292 - 0.01954/L	0.002689	0.1537
188.	7.50	8 I	18.4	4.65	0.01287	0.01608 - 0.01494/L	0.001879	0.1056

Table 3-4 (Continued)
RESISTANCE FUNCTIONS FOR UNIFORMLY-LOADED STEEL BEAMS, ONE END FIXED AND ONE END SIMPLY-SUPPORTED

	L_{fv} ft.	Shape		L_{ep} ft.	$\frac{q_v B L}{f_{dy}}$ $(L < L_{ep})$	$\frac{q_v B L}{f_{dy}}$ $(L_{ep} < L < L_{fv})$	$\frac{q B}{f_{dy}}$ $(L = L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L > L_{fv})$
189.	6.99	10 B	17	4.05	0.01513	0.01892 - 0.01533/L	0.002393	0.1169
190.	6.98	14 B	17.2	3.65	0.01887	0.02359 - 0.01720/L	0.003027	0.1474
191.	6.92	12 I	50	3.88	0.04893	0.06117 - 0.04749/L	0.007847	0.3758
192.	6.40	8 B	15	3.83	0.01227	0.01533 - 0.01174/L	0.002108	0.08644
193.	6.40	12 B	16.5	3.43	0.01760	0.02200 - 0.01509/L	0.003071	0.1256
194.	6.20	12 B	14	3.29	0.01527	0.01908 - 0.01257/L	0.002750	0.1058
195.	6.13	10 B	15	3.44	0.01453	0.01817 - 0.01249/L	0.002631	0.09889
196.	6.04	8 B	10	3.57	0.008467	0.01058 - 0.007564/L	0.001545	0.05637
197.	5.94	10 B	11.5	3.31	0.01133	0.01417 - 0.009366/L	0.002119	0.07479
198.	5.81	8 M	20	3.42	0.01720	0.02150 - 0.01470/L	0.003266	0.1102
199.	5.68	12 Jr	11.8	2.92	0.01340	0.01675 - 0.009771/L	0.002646	0.08537
200.	5.44	10 I	35	2.99	0.03573	0.04467 - 0.02669/L	0.007312	0.2162
201.	5.40	6 B	8.5	3.31	0.006200	0.00775 - 0.005135/L	0.001260	0.03670
202.	5.17	8 I	23	2.98	0.02100	0.02625 - 0.01566/L	0.004491	0.1201
203.	4.34	7 I	20	2.48	0.01867	0.02333 - 0.01158/L	0.004763	0.08966

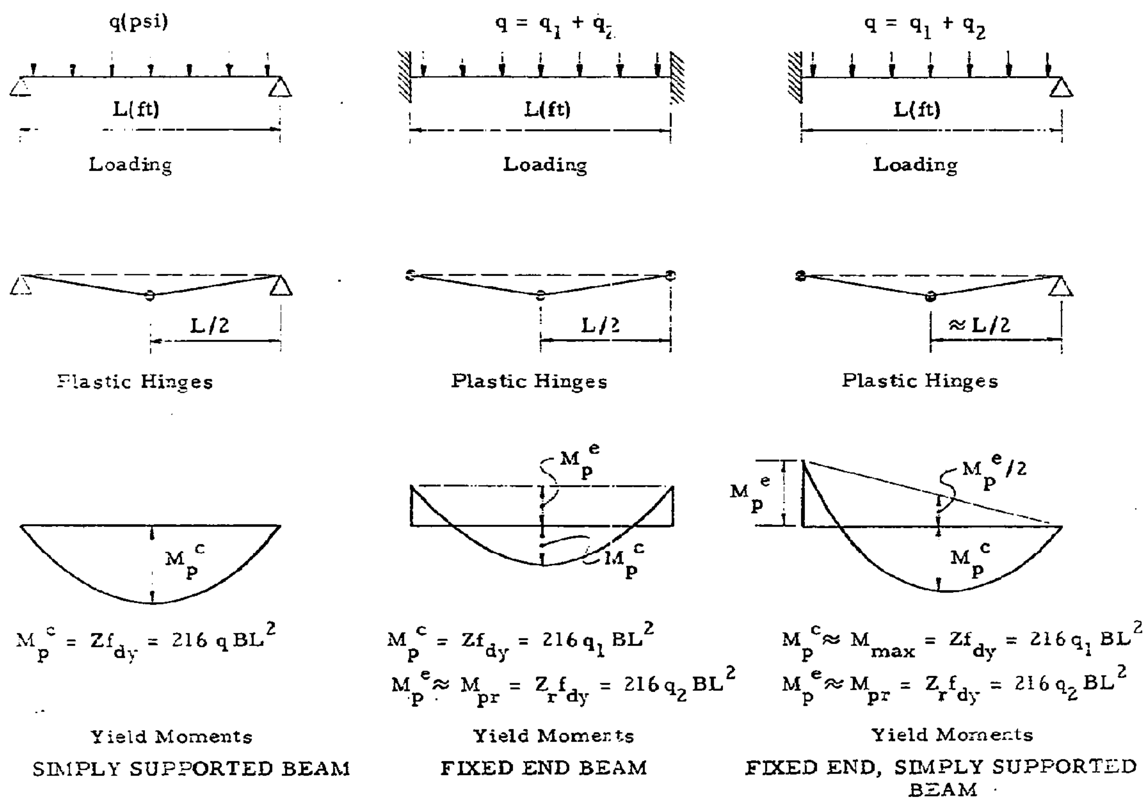


Figure 3-3

LIMITING FLEXURAL CAPACITIES FOR STEEL BEAMS, SPACING B FEET

where

b = flange width, (in.)

t_f = flange thickness, (in.)

d = web depth, (in.)

t_w = web thickness, (in.)

L_{cr} = unbraced length on one side of plastic hinge, (ft)

M = moment at end of L_{cr} away from hinge, (in.-lb)

r_g = radius of gyration (weak direction) of compression flange, (in.)

The in-place cost of a rolled beam can be directly determined from its weight and the unit steel costs listed in Chapter 2. Obviously, if shearing stresses govern the selection of a beam, and if a constant cost-per-pound is assumed for steel, the relative cost-efficiency of a section can be expressed as A_w/A. Since the unit weight of a steel beam is proportional to its gross cross-sectional area, and since its shearing resistance is proportional to the net web, the relative cost-efficiency can also be expressed in terms of the unit weight and shear resistance function.

$$C_s = w X_s \tag{3.23.13}$$

$$C_s = \left[\frac{q_v B L}{f_{dy}}\right] \cdot \left[\frac{f_{dy}}{q_v B L}\right] w X_s \tag{3.23.14a}$$

$$C_s \left[\frac{f_{dy}}{q_v B L}\right] = \frac{w f_{dy}}{q_v B L} X_s \tag{3.23.14b}$$

where

C_s = in-place cost per lineal foot of beam, (\$/ft)

w = weight per foot of beam, (lb/ft)

X_s = unit in-place cost of steel, (\$/lb)

Equation 3.23.14b indicates that the cost of a given beam per unit of shearing resistance is directly proportional to the shear cost function

$$\frac{w\, f_{dy}}{q_v\, B\, L} X_s ,$$

assuming that shear controls the design of the beam. Table 3-5 lists values of this function for selected standard beam sections.

3.24 Rectangular Bent

The rectangular steel bent is treated as a separate structural element. As visualized, such bents are fabricated from rolled steel sections with stiffener plates provided at the beam-to-column connections. The beam and the column are discussed as distinct structural elements in Sections 3.22 and 3.23, where equations are supplied for their design.

It is assumed that lateral earth support will prevent any side sway of the loaded bent. The beam which forms the horizontal bent member will support vertical loads and will also carry axial thrust from the column reactions. Similarly, the columns will support the vertical beam reactions and will also resist lateral loads. The analysis of the bent commences with the selection of the optimum beam section of length L which, for beam spacing B, can just support the uniformly-applied equivalent loading q. Next, a column section is selected which is just adequate to support the beam. Preliminary studies of typical bents indicate that, for the range of loading and span lengths considered in this analysis, the moment and thrust in the bent columns can be approximated by:

1) Computing beam end moments on the assumptions of fixed ends and clear (column-face to column-face) spans.

Table 3-5

SHEAR RESISTANCE AND SHEAR COST FUNCTIONS FOR SELECTED ROLLED STEEL SECTIONS

Shape	Shear Resistance Function $\frac{q_v B L}{f_{dy}}$	Shear Cost Function $\frac{f_{dy} w}{q_v B L}$
36 WF 300	.2627	114.2
36 WF 280	.2460	113.8
36 WF 260	.2349	110.5
36 WF 245	.2230	109.8
36 WF 194	.2179	89.2
36 WF 182	.2052	88.8
36 WF 170	.1924	88.4
36 WF 160	.1848	86.7
36 WF 150	.1768	84.9
36 WF 135	.1693	79.8
33 WF 130	.1518	85.8
24 I 120	.1450	82.8
33 WF 118	.1449	81.5
14 I 100	.1386	72.3
30 WF 99	.1231	79.5
20 I 95	.1211	78.4
24 I 90	.1158	77.8
27 WF 94	.1038	90.6
20 I 85	.0988	86.0
20 I 75	.0984	76.2
27 WF 84	.0981	97.1
24 WF 76	.0827	91.7
24 WF 68	.0782	87.0
20 I 65.4	.0768	85.2
21 WF 62	.0658	94.3
18 I 54.7	.0637	85.9
15 I 50	.0631	79.2

Table 3-5 (Continued)

Shape	Shear Resistance Function $\frac{q_v B L}{f_{dy}}$	Shear Cost Function $\frac{f_{dy} w}{q_v B L}$
12 I 50	.0612	81.8
18 WF 55	.0548	100.0
18 WF 50	.0503	99.4
15 I 42.5	.0470	90.5
10 I 35	.0447	78.4
16 WF 40	.0385	104.0
16 B 31	.0343	90.4
16 B 26	.0312	83.5
14 WF 30	.0295	101.8
14 B 22	.0250	88.0
14 B 172	.0236	73.0
12 B 16.5	.0220	75.1
10 B 15	.0182	82.5
12 Jr 11.8	.0168	70.3
10 Jr 9	.0124	72.6
8 Jr 6.5	.0086	75.5

2) Equating column end-moments to beam end-moments. Thus, any moment due to eccentricity of beam shear at the column connection is neglected.

3) Equating the axial column loading to the beam shear at the column face.

The beam analysis of Section 3.23 postulates that the plastic moment capacity at the end of a fixed-end beam should be taken as M_{pr} rather than as M_p, thus making ample provision for combined flexural and shear stresses. Extending this same reasoning, a reduced plastic moment capacity M_{pr} is used in analyses of the column members. This, for those cases where design is not controlled by shearing stresses, results in some excess moment capacity at the beam-column joint. However, since the beam must also support an axial thrust from the column reactions, a moderate degree of conservatism appears to be justified.

The bent analysis considers the relation between end moment and end shear (M/V ratio) for the horizontal member of a rectangular bent, as the length of the beam is increased. The beam is considered to be loaded to its ultimate capacity, which is controlled by web shear for $L < L_{fv}$ and by total plastic moment capacity for $L > L_{fv}$. The vertical shear at the beam-column connection is constant for $L < L_{fv}$, then decreases for $L > L_{fv}$. The moment at the end of the beam increases until $L = L_{ep}$, when extreme-fiber yielding occurs at the first incipient plastic hinge. For $L_{ep} < L < L_{fv}$, which describes an elasto-plastic range where yield hinges are forming in regions of maximum flexural stress, the end moment changes from the maximum elastic moment (M_e) to the reduced plastic moment (M_{pr}).

The difference between M_e and M_{pr} is small for most standard rolled sections and, as a useful design simplification, it is assumed that the beam end-moment remains constant through its elasto-plastic range. Finally, when the beam has developed the full moment capacity of its plastic hinges at $L = L_{fv}$, the end-moment remains at a constant value of M_{pr} for increasing values of L.

Thus, there are three M/V ranges of interest. For $L < L_{ep}$, M is increasing and V is constant. For $L_{ep} < L < L_{fv}$, both M and V are

approximately constant. For $L > L_{fv}$, M is constant and V is decreasing. The design column loading, applying the previously stated assumptions, is the sum of beam loading (M/V ratio) and the direct load on the column itself. This latter term is a function of loading intensity and column width, hence cannot be expressed explicitly. Since it may be of significance for short bent spans when the loading intensity is large, it is included in the general analysis.

The strength of an eccentrically-loaded column, with M_{pr} substituted for M_p, is obtained from Equation 3.21.2.

$$\frac{P'_{dy}}{A\,f_{dy}} = 1 - 0.85\left[\frac{M'_p}{M_{pr}}\right] - \frac{\alpha H}{5000\, r_g}\sqrt{f_{dy}} \qquad (3.24.1)$$

For the anticipated conditions of span and load, it makes little practical difference whether column bases are assumed fixed, pinned or partially restrained. By taking $\alpha = 1.0$, $H = 8$ ft and $f_{dy} = 50{,}000$ psi, which represent typical values for the structural steels and cubicle designs considered in this study, Equation 3.21.2 can be written for the column members of the bent as,

$$\frac{P'_{dy}}{f_{dy}\, A_{column}} = 1.00 - 0.85\left[\frac{M'_p}{M_{pr}}\right] - \frac{1}{2.80\, r_g} \qquad (3.24.2)$$

In this equation, P'_{dy} is the axial load on the column. M'_p is the moment which is applied to the column, and is assumed equal to the end moment of the beam. Thus, depending on the relation between beam span and loading, the moment M'_p applied to the column may have a value which is less than, or equal to, the reduced plastic moment M_{pr} for the beam.

Solving Equation 3.24.2 in terms of the column loading P'_{dy}, yields

$$P'_{dy} = \frac{M_{pr}\left[\frac{2.80\, r_g - 1}{2.80\, r_g}\right]}{\left[\frac{M_{pr}}{f_{dy} A_{column}}\right] + 0.85\left[\frac{M'_p}{P'_{dy}}\right]} \qquad (3.24.3)$$

The column load can also be expressed as the sum of beam end-shear and the direct load on the column.

$$P'_{dy} = V_{beam} + 12\,q\,B\,D = 72\,q\,B\,L\left[1 + \frac{d_{column}}{6\,L_{beam}}\right]$$

where

D = gross depth of column section, (in.)
Also, if moment and thrust are such that the full compressive and flexural capacities of the column are developed, then $M'_p = M_{column}$ and $P'_{dy} = P_{column}$.

Substituting in Equation 3.24.3 yields

$$\frac{q\,B\,L}{f_{dy}} = \left[\frac{L\left[2.80 - \frac{1}{r_g}\right]}{\frac{33.6(D_{beam} + 6\,L)}{A_{column}} + \frac{28.6}{Z_{r\ column}}\left\{(6\,L + D)\frac{M_{column}}{P_{column}}\right\}}\right] \qquad (3.24.4)$$

M_{column} is the end moment of a fixed-end beam of length L and spacing B which supports a unit load q. P_{column} is the end shear from this beam, plus the load on a column section of width D due to the same spacing B and unit load q. The ratio of M_{column} to P_{column} can thus be expressed in terms of the M/V ratio of the horizontal bent member, which is in turn related to its characteristic lengths.

$$0 < L \leq L_{ep} \qquad \frac{M_{column}}{P_{column}} = \frac{12\,L^2}{6\,L + D_{column}}$$

$$L_{ep} \leq L \leq L_{fv} \qquad \frac{M_{column}}{P_{column}} = \frac{12\,L_{ep}}{6\,L + D_{column}} \qquad (3.24.5)$$

$$L > L_{fv} \qquad \frac{M_{column}}{P_{column}} = \frac{8\,L^2}{6\,L + D_{column}}$$

Substituting Equation 3.24.5 into Equation 3.24.4, inverse column resistance functions are obtained in terms of the characteristic lengths L_{ep} and L_{fv} of the horizontal bent member and the column coefficients K_1, K_2, K_3, of the vertical bent members.

$$0 < L \leq L_{ep} \qquad \frac{f_{dy}}{q\,B\,L} = \frac{K_1}{L} + K_2 + K_3\,L$$

$$L_{ep} \leq L \leq L_{fv} \qquad \frac{f_{dy}}{q\,B\,L} = \frac{K_1}{L} + K_2 + K_3\,L_{ep} \qquad (3.24.6)$$

$$L > L_{fv} \qquad \frac{f_{dy}}{q\,B\,L} = \frac{K_1}{L} + K_2 + 0.667 K_3\,L$$

where

$$K_1 = \frac{33.6\,D_{column}}{A_{column}\left(2.80 - \frac{1}{r_g}\right)}$$

$$K_2 = \frac{202}{A_{column}\left(2.80 - \frac{1}{r_g}\right)}$$

$$K_3 = \frac{344}{Z_{r\ \text{column}} \left(2.80 - \frac{1}{r_g} \right)}$$

$$L_{ep} = \frac{Z_{r\ \text{beam}}}{1.20\ A_{w\ \text{beam}}}$$

$$L_{fv} = \frac{(Z_{\text{beam}} + Z_{r\ \text{beam}})}{1.80\ A_{w\ \text{beam}}}$$

Table 3-6 lists values of K_1, K_2 and K_3 for standard column sections. For convenient reference, the beam resistance functions for the sections are listed in the same table. To use the table, first employ the known values of q, B and L to select a suitable beam section for the horizontal bent member. Next, find a section whose column resistance function is equal to or slightly greater than the beam resistance function of the transverse bent member. Obviously, the column and beam resistance functions are both influenced by the relation between L, L_{ep}, and L_{fv}. The load imposed on the column by the beam is found to be a maximum when $L = L_{ep}$. For many cases of practical interest $L < L_{ep}$, hence, shear controls and the loaded beam is in its elastic range. The design of a bent with equal beam and column strengths can be facilitated by plotting beam and column resistance functions as functions of the bent span. In this way a graphic solution can readily be obtained for the least-weight combination of beam and column.

Limiting situations have been examined to assess the effect of variations in column height, degree of base fixity for the column, axial thrust transmitted to the transverse bent member, and lateral earth loading. These studies indicate that the beam and column resistance functions of Table 3-6 can be used, within an estimated accuracy range of $\pm$ 10 per cent, to select bent members for buried shelters which satisfy the following conditions:

1) No side sway or column buckling.

2) Column height not to exceed two stories.

3) Minimum bent span approximately equal to story height.

Table 3-6

DESIGN COEFFICIENTS FOR UNIFORMLY-LOADED RECTANGULAR STEEL BENTS

	Shape	Beam Characteristics for Section				Column Coefficients		
		L_{ep} ft.	L_{fv} ft.	$\frac{q_v B L}{f_{dy}}$ $(L \le L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L \ge L_{fv})$	K_1	K_2	K_3
1.	36 WF 300	26.21	39.59	0.2627	10.398	5.118	0.8363	0.1264
2.	36 WF 280	26.00	39.30	0.2460	9.668	5.450	0.8958	0.1361
3.	14 WF 426	27.97	39.05	0.1973	7.703	1.883	0.6046	0.1620
4.	14 WF 398	27.32	38.18	0.1862	7.109	1.977	0.6478	0.1759
5.	36 WF 260	24.86	37.79	0.2349	8.877	5.819	0.9634	0.1491
6.	14 WF 370	26.78	37.46	0.1741	6.522	2.084	0.6971	0.1920
7.	36 WF 245	24.41	37.19	0.2230	8.292	6.155	1.024	0.1600
8.	36 WF 230	23.80	36.37	0.2127	7.734	6.514	1.089	0.1721
9.	14 WF 314	25.90	36.29	0.1488	5.401	2.357	0.8227	0.2327
10.	33 WF 240	23.57	35.69	0.2123	7.579	5.851	1.048	0.1743
11.	14 WF 287	25.16	35.30	0.1378	4.866	2.523	0.9007	0.2587
12.	14 WF 264	24.89	34.95	0.1268	4.429	2.694	0.9795	0.2845
13.	33 WF 220	22.90	34.80	0.1983	6.898	6.328	1.142	0.1922
14.	14 WF 246	24.63	34.59	0.1183	4.093	2.849	1.052	0.3082
15.	14 WF 228	24.36	34.23	0.1099	3.763	3.027	1.135	0.3356
16.	14 WF 237	24.34	34.21	0.1147	3.923	2.934	1.092	0.3218
17.	30 WF 210	22.67	34.08	0.1793	6.110	6.073	1.199	0.2153
18.	33 WF 200	22.32	34.02	0.1829	6.223	6.916	1.258	0.2138
19.	14 WF 219	24.16	33.97	0.1058	3.592	3.129	1.183	0.3518
20.	14 WF 184	23.90	33.62	0.08833	2.970	3.612	1.409	0.4261
21.	14 WF 202	23.89	33.60	0.09783	3.288	3.341	1.282	0.3848
22.	14 WF 211	23.76	33.43	0.1031	3.446	3.221	1.227	0.3671
23.	30 WF 190	22.12	33.35	0.1642	5.475	6.655	1.326	0.2409
24.	14 WF 193	23.69	33.35	0.09367	3.123	3.469	1.343	0.4053
25.	14 WF 158	23.24	32.75	0.07683	2.516	4.103	1.641	0.5042
26.	14 WF 176	23.24	32.74	0.08625	2.824	3.745	1.474	0.4490
27.	14 WF 150	23.05	32.48	0.07308	2.374	4.292	1.731	0.5347
28.	14 WF 167	22.99	32.41	0.08208	2.661	3.914	1.553	0.4769
29.	30 WF 170	21.40	32.39	0.1515	4.907	7.288	1.463	0.2699
30.	24 WF 160	21.55	31.85	0.1228	3.910	6.530	1.585	0.3328

Table 3-6 (Continued)

DESIGN COEFFICIENTS FOR UNIFORMLY-LOADED RECTANGULAR STEEL BENTS

	Shape	Beam Characteristics for Section				Column Coefficients		
		L_{ep} ft.	L_{fv} ft.	$\frac{q_v B L}{f_{dy}}$ $(L \leq L_{fv})$	$\frac{q_v B L^2}{f_{dy}}$ $(L \geq L_{fv})$	K_1	K_2	K_3
31.	14 WF 142	22.12	31.24	0.07150	2.234	4.483	1.824	0.5698
32.	14 WF 127	21.81	30.83	0.06417	1.978	4.983	2.045	0.6441
33.	27 WF 177	20.50	30.79	0.1506	4.637	6.495	1.427	0.2844
34.	24 WF 145	20.72	30.74	0.1138	3.497	7.142	1.750	0.3736
35.	27 WF 160	20.43	30.71	0.1367	4.197	7.134	1.581	0.3143
36.	14 WF 119	21.69	30.67	0.06000	1.841	5.274	2.182	0.6926
37.	14 WF 136	21.63	30.61	0.06942	2.124	4.693	1.909	0.6001
38.	14 WF 103	21.50	30.43	0.05208	1.585	5.996	2.525	0.8053
39.	14 WF 111	21.32	30.18	0.05683	1.715	5.603	2.340	0.7444
40.	27 WF 145	19.98	30.10	0.1247	3.753	7.806	1.742	0.3524
41.	14 WF 87	21.16	29.97	0.04417	1.324	6.979	2.991	0.9657
42.	14 WF 95	21.02	29.78	0.04892	1.457	6.438	2.736	0.8775
43.	24 WF 130	19.58	29.22	0.1057	3.088	7.892	1.953	0.4258
44.	21 WF 142	19.41	28.57	0.1058	3.023	6.421	1.795	0.4308
45.	21 WF 127	19.36	28.49	0.09442	2.690	7.109	2.008	0.4843
46.	12 WF 190	20.19	28.43	0.09633	2.740	3.291	1.373	0.4656
47.	36 WF 194	17.37	27.88	0.2179	6.076	7.858	1.292	0.2302
48.	21 WF 112	18.80	27.74	0.08467	2.349	7.972	2.278	0.5564
49.	12 WF 161	19.62	27.67	0.08225	2.276	3.751	1.621	0.5621
50.	36 WF 182	17.19	27.64	0.2052	5.671	8.346	1.379	0.2471
51.	36 WF 170	16.98	27.36	0.1924	5.264	8.902	1.477	0.2668
52.	24 WF 120	17.80	26.86	0.1040	2.793	8.568	2.115	0.4758
53.	12 WF 133	18.94	26.77	0.06867	1.839	4.386	1.967	0.6981
54.	33 WF 152	16.80	26.76	0.1661	4.444	9.236	1.654	0.3130
55.	24 WF 110	17.71	26.73	0.09542	2.551	0.288	2.307	0.5215
56.	14 WF 84	18.67	26.66	0.04742	1.264	7.313	3.094	1.020
57.	36 WF 160	16.34	26.50	0.1848	4.898	9.408	1.568	0.2887
58.	24 WF 100	17.37	26.30	0.08758	2.302	10.15	2.537	0.5791
59.	14 WF 78	18.08	25.86	0.04500	1.164	7.814	3.954	1.350
60.	14 WF 320	18.05	25.82	0.1988	5.135	2.265	0.8086	0.2503
61.	18 WF 114	17.60	25.76	0.08185	2.109	6.936	2.252	0.6184

Table 3-6 (Continued)

DESIGN COEFFICIENTS FOR UNIFORMLY-LOADED RECTANGULAR STEEL BENTS

	Shape	Beam Characteristics for Section				Column Coefficients		
		L_{ep} ft.	L_{fv} ft.	$\frac{q_v B L}{f_{dy}}$ $(L \leq L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L \geq L_{fv})$	K_1	K_2	K_3
62.	33 WF 141	15.97	25.66	0.1583	4.060	9.894	1.782	0.3455
63.	36 WF 150	15.68	25.63	0.1768	4.533	9.989	1.672	0.3143
64.	12 WF 106	17.85	25.31	0.05633	1.426	5.303	2.470	0.9044
65.	12 WF 120	17.81	25.27	0.06450	1.630	4.768	2.181	0.7912
66.	18 WF 105	17.22	25.24	0.07617	1.923	7.469	2.446	0.6796
67.	36 WF 135	13.81	25.14	0.1693	3.915	11.03	1.861	0.3733
68.	12 WF 99	17.69	25.08	0.05275	1.524	5.631	2.650	0.9753
69.	12 WF 85	17.65	25.05	0.04500	1.127	6.433	3.088	1.147
70.	18 WF 96	16.89	24.82	0.07042	1.748	8.098	2.676	0.7494
71.	12 WF 92	17.42	24.73	0.04950	1.225	5.994	2.850	1.056
72.	16 WF 96	16.85	24.50	0.06500	1.592	7.318	2.690	0.8182
73.	12 WF 79	17.10	24.32	0.04275	1.040	6.857	3.323	1.246
74.	12 WF 72	16.94	24.10	0.03908	0.9418	7.449	3.648	1.377
75.	33 WF 130	14.79	24.08	0.1518	3.654	10.67	1.934	0.3893
76.	30 WF 132	15.02	23.95	0.1450	3.474	9.648	1.910	0.4022
77.	12 WF 65	16.75	23.85	0.03542	0.8447	8.164	4.042	1.537
78.	16 WF 88	16.16	23.56	0.06117	1.442	7.908	2.936	0.9071
79.	30 WF 124	14.59	23.39	0.1380	3.228	10.23	2.035	0.4349
80.	10 WF 112	16.48	23.21	0.05592	1.298	4.492	2.369	0.9988
81.	12 WF 58	16.12	23.01	0.03267	0.7518	9.198	4.527	1.731
82.	14 WF 74	15.77	22.79	0.04733	1.079	8.317	3.517	1.210
83.	27 WF 114	14.42	22.77	0.1208	2.748	10.09	2.219	0.5045
84.	10 WF 100	15.96	22.52	0.05075	1.143	4.916	2.652	1.138
85.	14 WF 68	15.50	22.43	0.04400	0.9867	8.969	3.827	1.325
86.	30 WF 116	13.81	22.34	0.1330	2.972	10.87	2.174	0.4770
87.	27 WF 102	13.98	22.17	0.1097	2.431	11.19	2.480	0.5733
88.	14 WF 61	15.28	22.13	0.03975	0.8798	9.896	4.269	1.488
89.	33 WF 118	13.30	22.10	0.1449	3.203	11.68	2.133	0.4534
90.	10 WF 89	15.63	22.08	0.04550	1.004	5.411	2.984	1.297
91.	10 WF 77	15.31	21.65	0.03958	0.8569	6.108	3.451	1.524
92.	12 WF 53	15.06	21.59	0.03133	0.6764	9.965	4.957	1.934

Table 3-6 (Continued)

DESIGN COEFFICIENTS FOR UNIFORMLY-LOADED RECTANGULAR STEEL BENTS

	Shape	Beam Characteristics for Section				Column Coefficients		
		L_{ep} ft.	L_{fv} ft.	$\frac{q_v B L}{f_{dy}}$ $(L \le L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L \ge L_{fv})$	K_1	K_2	K_3
93.	10 WF 54	15.22	21.52	0.02725	0.5865	8.326	4.936	2.232
94.	10 WF 66	15.17	21.46	0.03383	0.7259	6.980	4.035	1.802
95.	10 WF 60	15.15	21.43	0.03075	0.6590	7.580	4.437	1.986
96.	27 WF 94	13.30	21.26	0.1038	2.206	12.08	2.693	0.6370
97.	24 WF 94	13.44	21.05	0.09692	2.040	10.95	2.704	0.6774
98.	10 WF 72	14.80	20.97	0.03775	0.7918	6.468	3.696	1.654
99.	10 WF 49	14.78	20.95	0.02517	0.5272	9.080	5.448	2.489
100.	30 WF 108	12.66	20.81	0.1293	2.690	11.61	2.337	0.5356
101.	18 WF 77	13.60	20.43	0.06533	1.335	10.11	3.340	1.004
102.	18 WF 85	13.60	20.41	0.07233	1.477	9.241	3.026	0.9072
103.	24 WF 84	12.91	20.35	0.08833	1.797	12.14	3.024	0.7738
104.	21 WF 96	12.99	20.00	0.09233	1.847	9.383	2.663	0.7392
105.	18 WF 70	13.24	19.95	0.06025	1.202	11.03	3.678	1.119
106.	18 WF 64	13.06	19.70	0.05542	1.092	11.98	4.023	1.234
107.	18 WF 45	13.20	19.53	0.03875	0.7569	17.02	5.718	1.747
108.	30 WF 99	11.70	19.53	0.1231	2.405	12.60	2.551	0.6086
109.	21 WF 82	12.58	19.45	0.08017	1.560	10.84	3.118	0.8797
110.	27 WF 84	11.90	19.40	0.09808	1.903	13.41	3.015	0.7536
111.	24 WF 76	12.10	19.27	0.08267	1.593	13.32	3.342	0.8823
112.	24 I 105.9	12.17	19.25	0.1135	2.185	9.659	2.415	0.6396
113.	14 WF 53	12.94	19.01	0.03892	0.7398	11.42	4.916	1.797
114.	16 WF 78	12.68	18.94	0.06425	1.217	9.023	3.317	1.102
115.	14 WF 48	12.66	18.64	0.03567	0.6647	12.51	5.435	2.005
116.	16 WF 71	12.46	18.63	0.05900	1.099	9.821	3.646	1.222
117.	21 WF 73	11.82	18.52	0.07492	1.387	12.39	3.500	1.001
118.	12 WF 50	12.69	18.44	0.03375	0.6222	10.68	5.258	2.132
119.	12 WF 40	12.68	18.42	0.02675	0.4927	13.09	6.576	2.694
120.	10 WF 45	12.87	18.40	0.02592	0.4767	9.998	5.928	2.778
121.	16 WF 64	12.19	18.27	0.05375	0.9821	10.79	4.047	1.371
122.	12 WF 45	12.50	18.18	0.03050	0.5545	11.75	5.844	2.396
123.	14 WF 43	12.28	18.13	0.03242	0.5879	13.83	6.065	2.274

Table 3-6 (Continued)

DESIGN COEFFICIENTS FOR UNIFORMLY-LOADED RECTANGULAR STEEL BENTS

	Shape		Beam Characteristics for Section				Column Coefficients		
			L_{ep} ft.	L_{fv} ft.	$\frac{q_v B L}{f_{dy}}$ $(L \le L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L \ge L_{fv})$	K_1	K_2	K_3
124.	21 WF	68	11.53	18.11	0.07083	1.284	13.22	3.753	1.086
125.	8 WF	67	12.76	18.01	0.03417	0.6153	6.067	4.045	2.157
126.	16 WF	58	11.88	17.87	0.04942	0.8831	11.81	4.467	1.530
127.	8 WF	48	12.65	17.85	0.02408	0.4300	8.024	5.664	3.098
128.	24 WF	68	10.87	17.62	0.07817	1.378	14.78	3.740	1.039
129.	21 WF	62	11.07	17.52	0.06583	1.153	14.42	4.122	1.217
130.	8 WF	58	12.22	17.29	0.03033	0.5245	6.824	4.679	2.542
131.	10 WF	39	12.02	17.26	0.02350	0.4057	11.34	6.845	3.283
132.	24 I	79.9	10.55	17.17	0.09275	0.1592	12.83	3.207	0.9027
133.	18 WF	60	11.05	17.08	0.05842	0.9979	13.04	4.287	1.382
134.	18 WF	55	10.62	16.51	0.05483	0.9051	14.11	4.673	1.534
135.	18 WF	50	10.38	16.18	0.05033	0.8146	15.43	5.144	1.710
136.	8 WF	35	11.36	16.14	0.01875	0.3026	10.54	7.786	4.445
137.	21 WF	55	9.99	16.07	0.06175	0.9920	16.11	4.648	1.439
138.	8 WF	40	11.30	16.06	0.02167	0.3479	9.368	6.813	3.863
139.	24 I	120	9.73	16.01	0.1450	2.322	8.528	2.132	0.6264
140.	16 WF	50	10.43	15.99	0.04750	0.7596	14.02	5.175	1.813
141.	16 WF	45	10.07	15.51	0.04325	0.6708	15.44	5.748	2.063
142.	10 WF	33	10.63	15.41	0.02158	0.3327	13.17	8.105	4.048
143.	8 WF	31	10.81	15.40	0.01717	0.2644	11.74	8.802	5.109
144.	16 WF	40	9.98	15.40	0.03850	0.5929	17.24	6.467	2.338
145.	12 WF	36	10.28	15.25	0.02833	0.4322	14.91	7.306	3.137
146.	12 WF	31	10.05	14.95	0.02467	0.3688	17.11	8.489	3.688
147.	14 WF	38	9.77	14.84	0.03417	0.5072	16.15	6.864	2.712
148.	24 I	90	8.58	14.53	0.1158	1.682	11.39	2.848	0.8905
149.	14 WF	34	9.35	14.29	0.03133	0.4477	17.90	7.671	3.091
150.	12 WF	27	9.48	14.19	0.02233	0.3169	19.36	9.721	4.322
151.	20 I	85	8.68	14.10	0.09883	1.394	10.14	3.043	1.038
152.	20 I	65.4	8.60	14.02	0.07675	1.076	13.18	3.954	1.350
153.	8 M	24	9.70	13.95	0.01450	0.2022	15.17	11.38	6.740
154.	8 WF	28	9.64	13.84	0.01692	0.2341	13.11	9.761	5.818

Table 3-6 (Continued)

DESIGN COEFFICIENTS FOR UNIFORMLY-LOADED RECTANGULAR STEEL BENTS

	Shape		Beam Characteristics for Section				Column Coefficients		
			L_{ep} ft.	L_{fv} ft.	$\frac{q_v B L}{f_{dy}}$ $(L \leq L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L \geq L_{fv})$	K_1	K_2	K_3
155.	16 WF	36	8.76	13.76	0.03733	0.5138	19.01	7.196	2.750
156.	8 WF	24	9.51	13.67	0.01458	0.1993	15.05	11.39	6.847
157.	10 WF	29	8.95	13.21	0.02217	0.2929	15.69	9.209	4.675
158.	14 WF	30	8.35	12.96	0.02950	0.3823	20.14	8.717	3.680
159.	24 I	100	7.33	12.86	0.1386	1.783	10.25	2.563	0.8712
160.	10 WF	25	8.67	12.84	0.01933	0.2483	17.97	10.69	5.534
161.	18 I	54.7	7.84	12.76	0.06367	0.8126	14.27	4.758	1.793
162.	8 M	34.3	8.75	12.66	0.02225	0.2817	10.63	7.975	4.878
163.	16 B	31	7.81	12.49	0.03425	0.4278	22.08	8.363	3.366
164.	20 I	95	7.23	12.15	0.1211	1.472	9.080	2.724	1.020
165.	6 M	25	8.43	11.94	0.01308	0.1562	11.41	11.41	8.975
166.	20 I	75	6.86	11.70	0.09842	1.152	11.50	3.450	1.321
167.	6 M	20	8.16	11.61	0.01092	0.1267	14.23	14.23	11.08
168.	15 I	42.9	7.27	11.60	0.04700	0.5452	15.33	6.133	2.648
169.	14 B	26	7.24	11.47	0.02775	0.3183	23.27	10.05	4.517
170.	6 WF	25	7.92	11.31	0.01458	0.1650	11.96	11.27	8.492
171.	6 WF	20	7.76	11.10	0.01175	0.1305	14.57	14.10	10.77
172.	16 B	26	6.64	10.93	0.03117	0.3406	26.04	9.984	4.360
173.	10 WF	21	7.17	10.84	0.01842	0.1997	21.01	12.73	7.045
174.	8 WF	20	7.16	10.57	0.01525	0.1611	18.55	13.67	8.697
175.	12 I	31.8	6.79	10.57	0.03183	0.3366	16.79	8.397	4.246
176.	12 I	40.8	6.67	10.38	0.04092	0.4248	13.15	6.574	3.365
177.	14 B	22	6.41	10.36	0.02500	0.2591	27.21	11.90	5.670
178.	10 I	25.4	6.46	9.87	0.02333	0.2303	17.83	10.70	6.182
179.	12 B	22	5.82	9.35	0.02483	0.2322	24.63	12.00	6.347
180.	18 I	70	5.25	9.31	0.09850	0.9172	11.15	3.718	1.736
181.	15 I	50	5.54	9.30	0.06308	0.5864	13.16	5.263	2.595
182.	8 WF	17	6.20	9.29	0.01417	0.1316	21.49	16.12	10.84
183.	12 I	35	5.64	9.04	0.03892	0.3518	15.28	7.638	4.189
184.	5 M	18.9	6.31	8.98	0.01075	0.09654	13.03	15.64	15.14
185.	8 M	17	5.86	8.84	0.01475	0.1303	21.50	16.12	11.01

Table 3-6 (Continued)

DESIGN COEFFICIENTS FOR UNIFORMLY-LOADED RECTANGULAR STEEL BENTS

	Shape		Beam Characteristics for Section				Column Coefficients		
			L_{ep} ft.	L_{fv} ft.	$\frac{q_v B L}{f_{dy}}$ $(L \leq L_{fv})$	$\frac{q_f B L^2}{f_{dy}}$ $(L \geq L_{fv})$	K_1	K_2	K_3
186.	6 WF	15.5	6.06	8.83	0.01092	0.09643	18.12	18.12	14.95
187.	10 B	19	5.61	8.79	0.01975	0.1736	24.00	14.05	8.404
188.	8 I	18.4	5.58	8.43	0.01608	0.1355	20.19	15.15	10.65
189.	12 B	19	5.12	8.41	0.02292	0.1928	28.05	13.84	7.834
190.	6 B	16	5.73	8.39	0.01175	0.09855	18.44	17.70	14.66
191.	10 B	17	4.86	7.80	0.01892	0.1475	26.73	15.85	10.13
192.	14 B	17.2	4.38	7.71	0.02358	0.1818	35.63	15.27	8.822
193.	12 I	50	4.66	7.70	0.06117	0.4708	10.73	5.363	3.237
194.	7 I	15.3	5.12	7.69	0.01292	0.09931	21.67	18.58	14.69
195.	8 B	15	4.59	7.17	0.01533	0.1099	24.68	18.24	13.54
196.	12 B	16.5	4.12	7.08	0.02200	0.1558	32.10	16.05	10.17
197.	6 I	12.5	4.66	6.95	0.01008	0.07007	23.34	23.34	21.15
198.	12 B	14	3.95	6.86	0.01908	0.1309	37.43	18.85	12.22
199.	10 B	15	4.13	6.82	0.01817	0.1239	29.99	17.99	12.47
200.	10 B	11.5	3.97	6.60	0.01417	0.09352	38.45	23.37	16.64
201.	8 M	20	4.10	6.49	0.02150	0.1396	18.37	13.78	10.85
202.	6 B	12	4.27	6.45	0.01058	0.06831	23.84	23.84	22.00
203.	8 B	13	3.94	6.29	0.01433	0.09015	28.21	21.16	16.95
204.	12 Jr	11.8	3.50	6.26	0.01675	0.1049	45.28	22.64	15.73
205.	5 I	10	4.19	6.19	0.007583	0.04695	25.33	30.39	32.40
206.	10 I	35	3.59	6.04	0.04467	0.2696	12.97	7.782	5.862
207.	8 I	23	3.58	5.77	0.02625	0.1514	16.18	12.14	10.23
208.	10 Jr	9	3.16	5.55	0.01242	0.06893	50.11	30.07	23.87
209.	7 I	20	2.98	4.84	0.02333	0.1128	16.63	14.25	14.12

4) Column bases fixed, pinned or partially restrained.

5) Lateral earth pressure coefficient k_h of any value less than one.

Because of the multitude of possible combinations, costs of rectangular bents are best determined by computing the costs of the individual bent members. The simplified design tables permit the rapid solution of a number of trial designs. These designs may be compared on the basis of cost by applying the unit cost coefficients presented in Chapter 2.

3.25 Segmented Bent

The segmented steel bents considered in this section are intended as ribs for equally-segmented arches. They are an attempt to approximate, without resorting to curved compression members, the condition of axial thrust which is assumed to exist in buried shell structures. The circular arch shape is more closely approximated as the number of bent segments is increased, but fabrication costs will also increase. As the number of segments is decreased, however, the thrust is no longer axial and moment in the bent segments becomes of increasing design importance.

The analyses assume that all segments are of equal lengths and cross-sections. As with the rectangular bent, B represents the center-to-center spacing, in feet, of the individual bents. The span length of the segmented bent, expressed in feet, is designated as S_L. Arch bents with four and six segments were initially considered, but subsequently rejected since the moments reached values which offered no real advantage over conventional rectangular bents. The eight-segmented arch, where $L' = 0.195\ S_L$, is found to represent a satisfactory and practical compromise. Analysis of a two-hinged arch with eight equal segments, subjected to uniform radial chord loading, yields the following results:

1) Maximum moment in the arch, which occurs in the haunch segment, is $7.10\ q\ B S_L^2$.

2) Maximum thrust in the haunch segment is $72\ q B S_L$.

3) Maximum shear in the haunch segment is $14.7\ q\ B S_L$.

4) Maximum thrust in the crown segment is $76.3\, q\, BS_L$.

5) Moment in arch at crown segment is $6.38\, q\, BS^2_L$.

Each segment of this arch would normally be analyzed as a compression member which carries moment and is susceptible to buckling about its strong axis. In this study, however, the inter-action of the arch configuration and the passive earth pressure is assumed to preclude a general buckling failure. The full yield strength of bent members will therefore be developed, since individual members will be short and stocky. The strength of an eccentrically-loaded steel compressive member is given by Equation 3.24.1 which, with its buckling term neglected, yields the expression,

$$\frac{P'_{dy}}{A f_{dy}} = 1 - 0.85 \frac{M'_p}{M_{pr}} \qquad (3.25.1)$$

Equation 3.25.1 can be applied to the segmented bent by substituting known values for the moment and thrust in the segments of the eight-segmented bent.

Haunch Segment
$$\frac{f_{dy}}{q_c\, BS_L} = \frac{72}{A} + \frac{6 S_L}{Z_r} \qquad (3.25.2)$$

Crown Segment
$$\frac{f_{dy}}{q_c\, BS_L} = \frac{76.3}{A} + \frac{5.4 S_L}{Z_r} \qquad (3.25.3)$$

Since the ratio of S_L/Z_r will be small for practicable sections, the design will frequently be governed by Equation 3.25.3. The critical shearing stresses at the haunch must also be checked. The required net area of web steel in the haunch section is given by

$$A_w\,(\text{required}) = 24.5 \frac{q\, BS_L}{f_{dy}} \qquad (3.25.4a)$$

Or, expressing this in terms of the inverse shear resistance functions for a given net area of web,

$$\frac{f_{dy}}{q\,B\,S_L} = \frac{24.5}{A_w} \qquad (3.25.4b)$$

The cost of segmented bents can be determined by applying the unit cost values given in Chapter 2 to a number of trial solutions for the segmented arch bent.

3.26 <u>Single-Curvature Plates in Compression.</u>

Single-curvature steel plate, either corrugated or of uniform thickness, is suitable for use in the arch or cylinder configuration. For this application, the plate is analyzed as a circular segment subjected to uniform radial load. It is again postulated that the passive earth resistance will prevent either a general flexure or buckling failure mode.

The design equation for a one-inch length of single-curvature steel plate, loaded in a compressive mode, is:

$$\frac{q_c\,S_L}{f_{dy}} = \frac{t}{6} \qquad (3.26.1)$$

where

q_c = unit compressive mode yield resistance, (psi)

S_L = diameter of arch or cylinder, (ft)

f_{dy} = dynamic yield strength of steel, (psi)

t = effective thickness of plate, (in.)

By substituting values of dynamic yield strength into this equation, the yield load can be calculated for unit lengths of standard corrugated plate. Yield loads are not tabulated for 100, 000 psi corrugated plate in gages heavier than No. 8, due to the limited capacity of existing press facilities.

Table 3-7

COMPRESSIVE YIELD CAPACITY FOR SINGLY-CURVED CORRUGATED STEEL PLATE, POUNDS PER LINEAL INCH

Gage No.	Effective Area, Sq. In.	Dynamic Yield Strength of Steel, psi 44,000	60,000	100,000
12	.1297	5,700	7,780	12,960
10	.1667	7,330	10,000	16,670
8	.2041	9,000	12,250	20,450
7	.2283	10,050	13,700	not available
5	.2666	11,720	16,000	" "
3	.3048	13,400	18,300	" "
1	.3432	15,100	20,600	" "

Tests of corrugated sections have indicated that the ultimate capacity of the 10 and 12 gage plates is a function of joint or seam strength, rather than of the material[36]. Thus, as corrugated steel culverts are currently fabricated, the values given in Table 3-7 for the 12 and 10 gage plates should be reduced by 30 percent and 5 percent respectively. However, it seems reasonable to assume that improved joining techniques could raise the ultimate capacity to that of the yield strength. The cost data supplied in Chapter 2 reflect this assumption. Typical joint details are explained in Reference 36.

Yield loads for uniform plates are computed for representative plate thicknesses by applying Equation 3.26.1. It is assumed that the joint strength which can be developed in flat plates will be equal to the yield strength of the metal.

Table 3-8

COMPRESSIVE YIELD CAPACITY FOR SINGLY-CURVED UNIFORM-THICKNESS STEEL PLATE, POUNDS PER LINEAL INCH

Thickness,	Dynamic Yield Strength of Steel, psi		
inches	44,000	60,000	100,000
0.75	33,000	45,000	75,000
0.50	22,000	30,000	50,000
0.25	11,000	15,000	25,000

The unit costs for singly-curved plate, as presented in Section 2.2, are expressed in $/sq ft of shell surface. The cost for the shell of a particular arch or cylinder can be determined by multiplying the unit cost of the selected plate by the surface area of the structure. Considerable savings can be realized by the use of high strength steel, as is indicated by the following tables.

Table 3-9

RELATIVE COST VERSUS RELATIVE COMPRESSIVE YIELD CAPACITY SINGLY-CURVED CORRUGATED STEEL PLATES

Plate	Dynamic Yield Strength psi	Relative Cost	Relative Strength
Corrugated	44,000	1.00	1.00
Corrugated	60,000	1.17	1.36
Corrugated	100,000	1.35	2.28

Table 3-10

RELATIVE COST VERSUS RELATIVE COMPRESSIVE YIELD CAPACITY FOR SINGLY-CURVED UNIFORM-THICKNESS STEEL PLATE

Plate	Dynamic Yield Strength psi	Relative Cost	Relative Strength
Uniform	44,000	1.00	1.00
Uniform	60,000	1.10	1.36
Uniform	100,000	1.27	2.28

3.27 Double-Curvature Plates in Compression

The double-curvature steel plate can be used in dome and sphere shelter configurations. The design considerations for double-curvature plate are, with one major exception, exactly the same as for single curvature plate. Since the doubly-curved plate is stressed biaxially, due to the two-way action of the loaded dome, the net result is that a doubly-curved plate can carry twice the unit load which the same area of singly-curved plate can support. Thus, for the doubly-curved plate, assuming that only axial stresses are effective,

$$\frac{q_c S_L}{f_{dy}} = \frac{t}{3} \qquad (3.27.1)$$

where

q_c = unit compressive mode yield resistance, (psi)

S_L = diameter of dome or sphere, (ft)

f_{dy} = dynamic yield strength of steel, (psi)

t = effective thickness of plate, (in.)

Since double-curvature corrugated plate is currently not available, this study does not contemplate its use in buried shelters. Table 3-11 supplies compressive yield capacities for uniform-thickness steel plate.

Table 3-11

COMPRESSIVE YIELD CAPACITY FOR DOUBLE-CURVATURE UNIFORM-THICKNESS STEEL PLATE, POUNDS PER LINEAL INCH

Thickness inches	Dynamic Yield Strength of Steel, psi 44,000	60,000	100,000
0.75	66,000	90,000	150,000
0.50	44,000	60,000	100,000
0.25	22,000	30,000	50,000

The general cost relationships which are discussed for single-curvature shell structures are also valid for doubly-curved plates. Unit costs of double-curvature uniform plate, expressed in $/sq ft of shell surface, are given in Chapter 2.

3.3 Reinforced Concrete

3.31 Introduction

The analyses of structural steel elements have considered only standard rolled shapes and plates, excluding built-up members and special fabrications. Steel elements are analyzed by selecting the rolled shapes or plates which, at least in-place cost, will furnish the required load resistance. Steel yield-strength is included as a variable in the structural steel evaluations, which increases the number of possible design solutions. However, simple evaluations of relative strength versus relative cost rapidly indicate the most economical design.

Several variables, in addition to strength of reinforcement steel, must be identified when reinforced concrete is studied for use as a structural material. In normal practice, a reinforced concrete element is uniquely designed and fabricated for its intended function. It thus becomes feasible to specify those structural forms and shapes which permit the most advantageous use of the material. Since the proportions and strength properties of the concrete constituents may themselves be varied, it is also desirable to establish the optimum material combination for each particular use.

In this study, the analyses of reinforced concrete elements commence by identifying the controlling modes of failure. These critical modes are then expressed, for a range of loading intensities, in terms of total required structural resistance. The relative contributions to total structural resistance, made by the concrete and by the reinforcing steel, are next evaluated. For each general case of loading, considering only the requirement for a specified ability to support load, several combinations of concrete and reinforcing steel are found to be adequate. Finally, by analyzing costs for each of the feasible material combinations, a minimum-cost design is obtained for each structural element.

The factors influencing the economics of the reinforced concrete elements are explored in considerable detail. Representative grades of concrete and reinforcing steel, with estimates of their in-place costs, are discussed in Chapter 2. The cost of form work is frequently found to be a major factor in establishing minimum-cost designs. By way of illustration, the cost of wooden forms will frequently represent over 50 percent of the total cost

of a reinforced concrete element. Unfortunately, the cost of form work cannot be estimated with the same precision as can the basic material costs.

3.32 Axially-Loaded Column or Bearing Wall

Axially-loaded columns of reinforced concrete can be used in buried group shelters as vertical members of a column-slab system, as interior columns in a continuous structure, or as exterior supporting members where significant moment is not transferred to the column from the floor, roof or walls. A unit length of an axially-loaded bearing wall may also be considered as a column, hence, the same design equations are applicable to the column and to the bearing wall. In the case of the wall, however, a minimum percentage of transverse steel is specified to provide for temperature-induced stresses.

The ultimate dynamic strength of reinforced concrete column or bearing wall, assuming an equivalent static load is applied axially, is expressed as[37],

$$P_{do} = 0.85\, f'_{dc}\, (A - A_s) + A_s f_{dy} \tag{3.32.1}$$

where

P_{do} = ultimate dynamic strength in direct compression of an axially-loaded reinforced concrete column, (lb)

f'_{dc} = unit compressive strength of concrete under dynamic loading, (psi)

A = cross-section area of column or of a unit length of bearing wall, (sq in.)

A_s = total cross-sectional area of main reinforcing steel in column or in a unit length of bearing wall, (sq in.)

Equation 3.32.1 is considered valid for $H/D \leq 1.25$ where

H = unsupported height of column, (ft)

D = gross column width in potential plane of bending, (in.)

Since the area of reinforcing steel is small in comparison with the gross column area for typical reinforced concrete columns, $(A - A_s) \approx A$. With this approximation,

$$P_{do} = 0.85 A f'_{dc} + A_s f_{dy} \tag{3.32.2}$$

Also, defining

$$p_t = \frac{A_s}{A}$$

and

$$q_{dt} = p_t \frac{f_{dy}}{f'_{dc}},$$

this becomes

$$\frac{P_{do}}{A f'_{dc}} = 0.85 + q_{dt} \tag{3.32.3}$$

The ultimate dynamic resistance of a column or bearing wall to axially-applied compressive loading, as indicated by Equation 3.32.1, is related to the cross-sectional areas of steel and concrete and to the maximum stresses which can be resisted by these materials. The ultimate dynamic strength of the concrete and the yield resistance of the reinforcing steel are assumed to combine linearly at ultimate column strength. Thus, an economic

evaluation of reinforced-concrete column design must examine the relative costs of concrete and of steel, as well as the contribution which each material makes to total column strength. A minimum percentage of steel reinforcement is specified, however, since some loading eccentricity is almost unavoidable. (It is recommended that reinforcement for columns be provided in the form of spiral-wound bars.) Bearing walls must also contain transverse temperature reinforcement, in addition to main reinforcing steel.

The unit costs of axially-loaded columns or bearing walls can be related to the cost factors C_c, C_s and C_f. The cost factor C_c refers to the cost of the concrete, per linear foot of column or per square foot of bearing wall surface. Cost factors C_s and C_f refer to costs of reinforcing steel and form work, respectively, in a unit of the structural element. The cost factor C_{st} refers to the cost of temperature or tie steel in a unit of the structural element. These cost factors can be expressed as

$$C_c \text{ (column)} = \left[\frac{A}{144}\right] X_c \tag{3.32.4a}$$

$$C_c \text{ (wall)} = \left[\frac{D}{12}\right] X_c \tag{3.32.4b}$$

$$C_s \text{ (column)} = \left[\frac{A}{144}\right]\left[\frac{\phi_t}{100}\right] X_s \tag{3.32.5a}$$

$$C_s \text{ (wall)} = \left[\frac{D}{12}\right]\left[\frac{\phi_t}{100}\right] X_s \tag{3.32.5b}$$

$$C_{st} \text{(column)} = \left[\frac{A}{144}\right]\left[\frac{\phi_{te}}{100}\right] X_s \tag{3.32.6a}$$

$$C_{st} \text{ (wall)} = \left[\frac{D}{12}\right]\left[\frac{\phi_{te}}{100}\right] X_s \tag{3.32.6b}$$

$$C_f \text{ (column)} = X_f \text{ (column) } P_r \tag{3.32.7a}$$

$$C_f \text{ (wall)} = X_f \text{ (wall)} \tag{3.32.7b}$$

where

X_c = unit cost of concrete, ($\$/ft^3$)

X_s = unit cost of steel, ($\$/ft^3$)

X_f = unit cost of form work, ($/sq ft)

P_r = perimeter of column, (ft)

ϕ_t = total area of main reinforcing steel expressed as percentage of gross concrete area, (normally 0.50%)

ϕ_{te} = transverse or temperature steel area expressed as percentage of gross concrete area, (normally 0.10%)

The composite cost factor for a wall or column, per square foot or linear foot respectively, can be expressed as,

$$C_t = C_c + C_s + C_{st} + C_f \qquad (3.32.8)$$

For the applications considered in this study, by assuming compressive axial loading and estimating in-place costs, it is found that the cost of the reinforcing steel in a concrete compression member is approximately five times the cost of the equivalent amount of concrete with the same load capacity. Thus, for reasons of economy, steel reinforcement in axially-loaded concrete compression members is kept to a minimum.

3.33 One-Way Reinforced Slabs and Beams

3.33.1 Design

One-way reinforced concrete beams and slabs can be used in the roof or floor of the cubicle. The ultimate resistance of all types of reinforced concrete slabs or beams can be expressed in terms of the possible modes of failure. Thus, for purposes of this study, the limiting resistances of the member in shear and in flexure are separately identified. Failure due to inadequate bonding of the reinforcing steel is also a possibility, but can be

controlled by proper detailing of reinforcement. Analyses of the shearing mode have considered the ability of a member to resist "pure" shear stresses, as well as its resistance to diagonal tension and/or shear compression.

The analyses of the possible failure modes for reinforced concrete beams or slabs assume that plastic yielding is initiated at locations of maximum moment as flexural stresses are increased. As plastification progresses at such sections, subsequent increases in flexural stresses will be transferred to other locations with reserve moment-resisting capacity. This process of stress readjustment and progressive plastic yielding will continue until a yield mechanism has developed in the member. The unit loading corresponding to this condition of incipient plastic collapse is considered as the ultimate flexural resistance for the slab or beam.

Axial thrust will frequently occur in combination with transverse loading, as in monolithic construction where a continuous roof slab supplies the reaction for lateral wall loading. However, selective analyses of the reinforced concrete beam-column show that this axial thrust, if limited to a fraction of the total vertical load on the member, will increase the flexural capacity of an under-reinforced beam-column. Thus, for the spans and types of loading anticipated in this study, the flexural capacity of concrete roof or floor slabs will not be reduced by axial thrust.

Computing equations for one-way reinforced slabs or beams with various conditions of end restraint are presented in the following paragraphs.

(1) Simply-Supported One-Way Beam or Slab

The maximum flexural stress will occur at mid-span. The plastic resisting moment of the reinforced cross section can be approximately expressed as

$$M_p = 0.009 \phi_c d^2 f_{dy} \tag{3.33.1}$$

Or, expressing M_p in terms of the unit flexural resistance and the loaded area of beam or slab,

$$\frac{q_f \times 12B \times 144L^2}{8} = 0.009\phi_c\, bd^2 f_{dy} \qquad (3.33.2)$$

where

q_f = unit flexural mode resistance, (psi)

B = center-to-center spacing of beams, (ft)
Note that, for the case of a slab, B has a value of 1/12 b.

L = span length of slab, (ft)

ϕ_c = tensile-steel percentage at mid-span, $\frac{100A'_s}{bd}$
A minimum value of $\phi_c = 0.25$ will be specified[1] for all flexural reinforced concrete members subjected to blast loading.

b = width of beam, (in.) or one-inch unit width for slab

d = effective depth of tension reinforcing steel, (in.)

Rearranging terms in Equation 3.33.2 results in the following.

$$\frac{q_f B}{b} = 0.0000416\phi_c \left(\frac{d}{L}\right)^2 f_{dy} \qquad (3.33.3)$$

The resistance of a simply-supported one-way reinforced beam or slab to diagonal tension and shear compression stresses, assuming no web reinforcement, can be expressed as[1],

$$\frac{q_{sc} \times 12B \times 144L^2}{b} = \frac{104}{(2+\theta')} d^2 (f'_c \phi_c)^{1/2} \qquad (3.33.4a)$$

where

q_{sc} = unit diagonal tension or shear compression mode resistance of beam or slab, (psi)

θ' = ratio of negative to positive reinforcement percentages at critical section
To make provision for possible stress reversals in blast loading, a minimum value of $\theta' = 0.25$ will be stipulated for all flexural reinforced-concrete members.

f'_c = unit static compressive strength of concrete (psi), based on the standard 28-day cylinder test.
Note the assumption that q_{sc} is limited by static concrete strength.

Rearranging terms,

$$\frac{q_{sc} B}{b} = \frac{0.0603}{(2+\theta')}\left(\frac{d}{L}\right)^2 \sqrt{f'_c \, \phi_c} \qquad (3.33.4b)$$

If web reinforcement is provided in the form of vertical stirrups, as is considered feasible in beams or slabs where d ≥ 10 in., the onset of diagonal tension cracking will be inhibited. The resistance of the member will then become[1],

$$\frac{q_{sc} B}{b} = \frac{0.0603}{(2+\theta')}\left(\frac{d}{L}\right)^2 (f'_c \, \phi_c)^{1/2} \left[1 + 0.00002\,\phi_v f_{dy}\right] \qquad (3.33.5a)$$

where

ϕ_v = percentage of web reinforcing steel

A minimum value of $\phi_v = 0.50\%$ is assumed for all cases where web reinforcement is employed(1).

Rearranging terms,

$$\frac{q_{sc} B}{b} = \left[\frac{0.0603 + 0.000001206\,\phi_v f_{dy}}{(2+\theta')}\right] (f'_c \phi_c)^{1/2} \left(\frac{d}{L}\right)^2 \qquad (3.33.5b)$$

The inclusion of compressive steel does not increase the flexural capacity of a simply-supported, under-reinforced beam or slab. Equation 3.33.5 also suggests that the resistance of the beam or slab in shear compression and diagonal tension will be reduced by an increase in θ', although this apparent decrease may not be real. The net result, however, is to favor the use of the minimum specified value for θ'. If so desired, Equation 3.32.5b and subsequent equations may be simplified by introducing $\theta' = 0.25$.

Considering the "pure" shear failure mode, the total shearing resistance of the cross-section of the beam or slab is expressed as[2]

$$V_u = 0.22\, b d f'_c \qquad (3.33.6)$$

where

V_u = ultimate shearing resistance of cross-section of beam or of unit width of slab, (lb)

The maximum shear will occur at the face of the support. Since the use of inclined bars[2] to resist shear has not been considered in this study, the shearing resistance of the concrete will control. The shearing resistance of the member is expressed in terms of static concrete strengths as

$$\frac{q_v \times 12\,B \times 12\,L}{2} = V_u = 0.22\, b d f'_c \qquad (3.33.7a)$$

where

q_v = unit shearing mode resistance, (psi)

Rearranging terms,

$$\frac{q_v B}{b} = 0.00306 f'_c \left(\frac{d}{L}\right) \qquad (3.33.7b)$$

Equations 3.33.3 to 3.33.7, inclusive, express the unit ultimate resistances of one-way reinforced concrete beams or slabs with simply-supported ends. By solving any two equations simultaneously, expressions can be obtained for balanced ultimate resistances in any two of these failure modes. In practice, the best procedure is to solve Equations 3.33.3 and 3.33.5b simultaneously, setting $q_f = q_{sc}$.

$$\frac{qB}{b} = 0.0000416\,\phi_c \left(\frac{d}{L}\right)^2 f_{dy}$$

$$= \left[\frac{0.0603 + 0.000001206\,\phi_v f_{dy}}{(2+\theta')}\right] (f'_c\,\phi_c)^{1/2} \left(\frac{d}{L}\right)^2 \qquad (3.33.8a)$$

Solving this expression for ϕ_c yields,

$$\phi_c = \left[\frac{1450 + 0.0289\,\phi_v f_{dy}}{f_{dy}\,(2+\theta')}\right]^2 f'_c \qquad \text{for } q_f = q_{sc} \qquad (3.33.8b)$$

The use of values of ϕ_c satisfying Equation 3.33.8b will, in theory, result in equal ultimate strengths in flexure and in diagonal tension or shear compression for simply-supported, one-way reinforced concrete slabs or beams. It is still possible that "pure" shear may control the design. However, this may be checked either by use of Equation 3.33.3 or by the plotted relationships between maximum q'_d and d/L, as shown in Figure 3-4, where

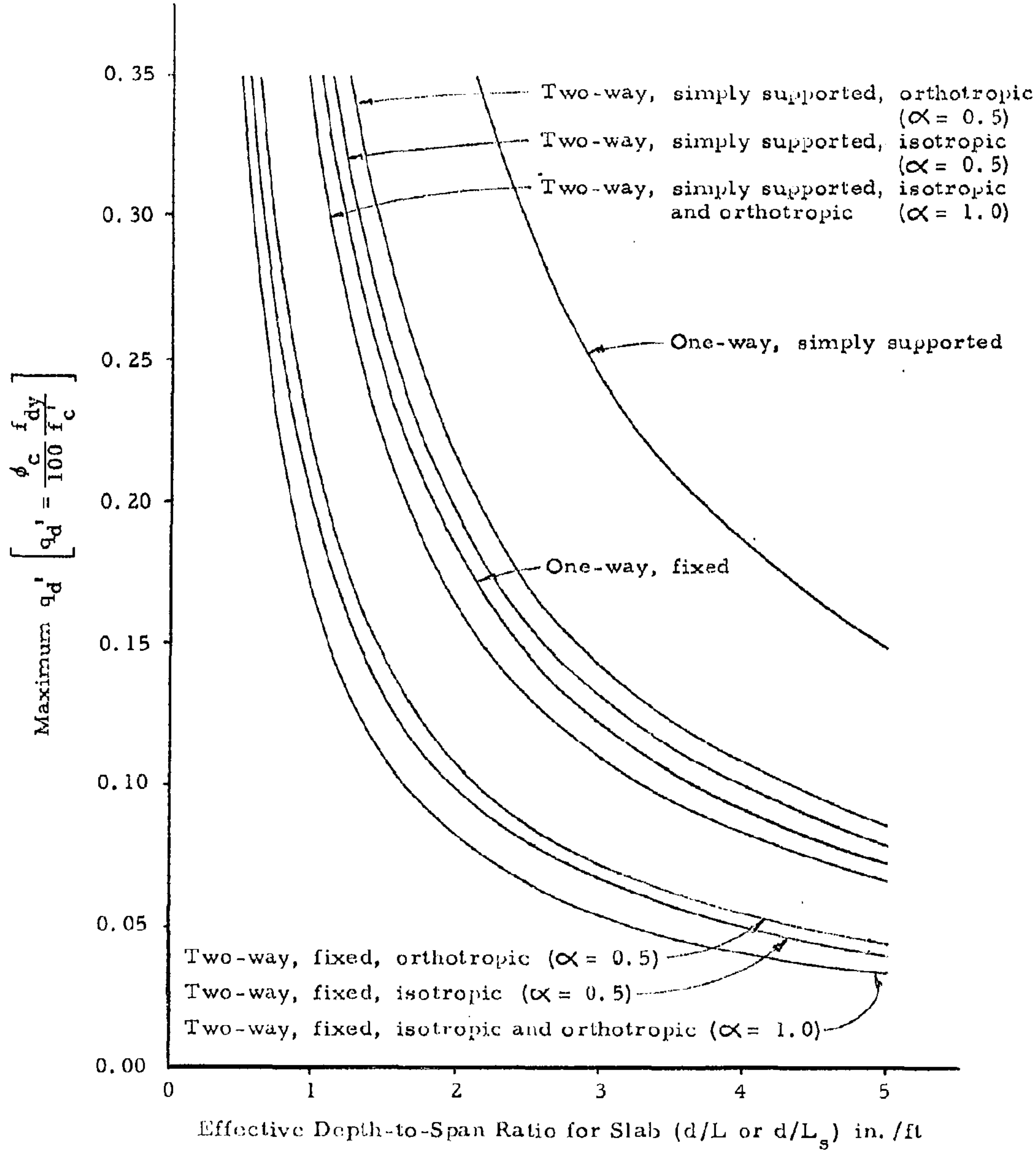


Figure 3-4

LIMITING COMBINATIONS OF q_d AND d/L FOR SLABS WHOSE ULTIMATE RESISTANCE IN FLEXURE (q_f) DOES NOT EXCEED THEIR ULTIMATE RESISTANCE IN PURE SHEAR (q_v)

$$q'_d = \frac{\phi_c}{100} \frac{f_{dy}}{f'_c}$$

The minimum value of 0.25 which is specified for ϕ_c will exclude the use of very lightly-reinforced sections. Also, since an abrupt failure is considered particularly undesirable, the maximum value of ϕ_c should be limited to ensure that the yield strength of the main reinforcing steel will govern the flexural resistance of the cross-section[26]. While the limiting values of ϕ_c corresponding to under-reinforced flexural sections are related to the ratio f'_c/f_{dy}, it has been suggested[1,2] that the maximum value of ϕ_c should not exceed 2 percent for structural grade (f_{dy} = 44 kips) reinforcement.

Table 3-12 can be used to obtain values of ϕ_c which, for specified levels of θ' and f_{dy} and for arbitrary levels of f'_c, result in $q_f = q_{sc}$. An acceptable value of ϕ_c must lie between the specified minimum and maximum limits. The resistance function $q L^2/\phi_c$ for a one-way reinforced slab can then be computed, combining the selected value of ϕ_c with the given values for q and L. The required depth of slab corresponding to this resistance function can then be read directly from the table. If a one-way reinforced beam of spacing B ft. is to be designed, the function $12 q B L^2/\phi_c$ must be computed and used in lieu of $q L^2/\phi_c$. After the depth of beam or slab has been determined from Table 3-12, a check must be made in order to ensure that "pure" shear will not control the design.

(2) Fixed-End One-Way Beam or Slab

The maximum flexural stresses in a fixed-end beam or slab will occur at mid-span and at the supports.

$$\frac{q_f \times 12 B \times 144 L^2}{8} = 0.009 (\phi_c + \phi_e) \, bd^2 f_{dy} \qquad (3.33.9a)$$

where

ϕ_e = effective tensile steel percentage at the supports

Table 3-12

RESISTANCE FUNCTIONS FOR ONE-WAY REINFORCED CONCRETE SLABS AND BEAMS, SIMPLY SUPPORTED

f_{dy} (psi)		44,000		52,000		60,000		75,000	
$\Theta' = \phi'/\phi_c$		0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
		Read value of k. Compute $\phi_c = f'_c/k$.							
$\phi_v = 0.0$		4660	8290	6510	11580	8670	15410	13540	24080
$\phi_v = 0.5$		2250	4010	2830	5020	3400	6040	4440	7890
$\phi_v = 1.0$		1320	2350	1[illegible]70	2790	1800	3200	2180	3870
$\phi_v = 1.5$		870	1550	1000	1770	1110	1970	1290	2290
Required Depth of Beam		Resistance Function $\frac{q_f L^2}{\phi_c} = \frac{q_{sc} L^2}{\phi_c}$ (psi-sq ft)							
D(in.)	d(in.)								
10	7.75	1320		1560		1800		2250	
12	9.50	1990		2350		2710		3380	
14	11.50	2910		3440		3970		4960	
16	13.50	4010		4740		5470		6830	
18	15.50	5290		6250		7210		9010	
20	17.50	6740		7960		9190		11480	
22	19.50	8370		9890		11410		14260	
24	21.50	10170		12020		13870		17330	
26	23.50	12150		14360		16570		20700	
28	25.50	14310		16910		19510		24400	
30	27.25	16340		19310		22300		27900	
32	29.25	18820		22200		25700		32100	
34	31.25	21500		25400		29300		36600	
36	33.25	24300		28800		33200		41500	
38	35.25	27300		32300		37300		46600	
40	37.25	30500		36100		41600		52000	
42	39.25	33900		40100		46200		57800	
44	41.25	37400		44200		51100		63800	
46	43.25	41200		48600		56100		70200	
48	45.25	45100		53200		61400		76800	
50	47.00	48600		57400		66300		82800	
52	49.00	52800		62400		72000		90000	
54	51.00	57200		67600		78000		97500	
56	53.00	61800		73000		84300		105300	
58	55.00	66600		78700		90800		113400	
60	57.00	71500		84500		97500		121800	

Rearranging terms,

$$\frac{q_f B}{b} = 0.0000416\,(\phi_c + \phi_e)\left(\frac{d}{L}\right)^2 f_{dy} \qquad (3.33.9b)$$

For this study, the percentages of positive reinforcement and of negative reinforcement for fixed-end slabs and beams are taken as equal, hence $\phi_c = \phi_e$. Equation 3.33.9b thus becomes,

$$\frac{q_f B}{b} = 0.0000833\,\phi_c \left(\frac{d}{L}\right)^2 f_{dy} \qquad (3.33.10)$$

The resistance of a fixed-end beam or slab to diagonal tension and shear compression, assuming no web reinforcement, can be expressed as[1]

$$\frac{q_{sc} \times 12\,B \times 144\,L^2}{b} = \frac{254}{(2+\theta')}\, d^2 \sqrt{f'_c\,\phi_c} \qquad (3.33.11a)$$

Rearranging terms,

$$\frac{q_{sc} B}{b} = \frac{0.147}{(2+\theta')}\left(\frac{d}{L}\right)^2 \sqrt{f'_c\,\phi_c} \qquad (3.33.11b)$$

Web reinforcement in the form of vertical stirrups will increase the resistance of the slab or beam to this mode of failure, as indicated in the following expressions:

$$\frac{q_{sc} B}{b} = \frac{0.147}{(2+\theta')}\left(\frac{d}{L}\right)^2 \sqrt{f'_c\,\phi_c}\left[1 + 0.00002\,\phi_v f_{dy}\right] \qquad (3.33.12a)$$

or

$$\frac{q_{sc}B}{b} = \left[\frac{0.147 + 0.0000294\,\phi_v f_{dy}}{(2+\Theta')}\right](f'_c\,\phi_c)^{1/2}\left(\frac{d}{L}\right)^2 \qquad (3.33.12b)$$

Equal areas of tension reinforcement are assumed for the top and bottom of the slab or beam ($\phi_e = \phi_c$), and a minimum ratio of compressive steel to tension steel ($\Theta' = 0.25$) is specified for all reinforced-concrete flexural members. Since there is no requirement for large open areas in hardened group shelters, economic considerations will undoubtedly favor the use of short spans. Evaluating these factors, in many cases of practical interest it is anticipated that areas of top and bottom steel in fixed-edge flexural members will be equal and remain constant throughout the length of the member ($\phi = \phi_e = \phi_c$, $\Theta' = 1.0$). This assumption could be introduced in Equation 3.33.12 and subsequent derivations, if so desired.

The resistance of the cross-section of the fixed-end beam or slab to "pure" shear is identical with that determined for the simply-supported beam or slab,

$$\frac{q_v B}{b} = 0.00306\,f'_c\left(\frac{d}{L}\right) \qquad (3.33.7b)$$

Thus, Equations 3.33.10, 3.33.12 and 3.33.7b express the ultimate resistances q_f, q_{sc}, and q_v for a one-way reinforced concrete beam or slab with fixed ends. Proceeding as explained for the simply-supported case, equating q_f and q_{sc} yields,

$$\phi_c = \left[\frac{1765 + 0.0353\,\phi_v f_{dy}}{f_{dy}\,(2+\Theta')}\right]^2 f'_c \qquad \text{for } q_f = q_{sc} \qquad (3.33.13)$$

Values of ϕ_c obtained from this equation must be checked by use of Equation 3.33.7b or Figure 3-4 to ensure that "pure" shear does not control. Also, ϕ_c must be kept within stipulated minimum and maximum limits, for the reasons previously described. Table 3-13 can be used to design fixed-end

Table 3-13

RESISTANCE FUNCTIONS FOR ONE-WAY REINFORCED CONCRETE SLABS AND BEAMS, BOTH ENDS FIXED

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_c$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_c = f'_c/k$.								
$\phi_v = 0.0$	3150	5590	4390	7810	5850	10400	9140	16250
$\phi_v = 0.5$	1520	2700	1900	3380	2290	4060	2990	5310
$\phi_v = 1.0$	890	1580	1060	1880	1210	2150	1460	2600
$\phi_v = 1.5$	580	1040	670	1190	750	1330	870	1540

Required Depth of Beam		Resistance Function $\frac{q_f L^2}{\phi_c} = \frac{q_{sc} L^2}{\phi_c}$ (psi-sq ft)			
D(in.)	d(in.)				
10	7.75	2640	3120	3600	4510
12	9.50	3970	4690	5420	6770
14	11.50	5820	6880	7940	9920
16	13.50	8020	9480	10940	13670
18	15.50	10570	12490	14420	18020
20	17.50	13480	15930	18380	23000
22	19.50	16730	19770	22800	28500
24	21.50	20300	24000	27700	34700
26	23.50	24300	28700	33100	41400
28	25.50	28600	33800	39000	48800
30	27.25	32700	38600	44600	55700
32	29.25	37700	44500	51300	64200
34	31.25	43000	50800	58600	73200
36	33.25	48700	57500	66300	82900
38	35.25	54700	64600	74600	93200
40	37.25	61100	72200	83300	104100
42	39.25	67800	80100	92400	115500
44	41.25	74900	88500	102100	127600
46	43.25	82300	97300	112200	140300
48	45.25	90100	106500	122900	153600
50	47.00	97200	114900	132500	165700
52	49.00	105600	124900	144100	180100
54	51.00	114400	135300	156100	195100
56	53.00	123600	146100	168600	211000
58	55.00	133100	157300	181500	227000
60	57.00	143000	169000	194900	244000

slabs and beams, following the procedures described for the simply-supported beam or slab. Its applicability is limited to cases where "pure" shear will not control the design.

(3) <u>One-Way Beam or Slab with One End Fixed, One End Simply-Supported</u>

The maximum flexural stresses in a beam or slab with one end fixed and one end simply-supported will occur at the fixed end and approximately at the center of the member.

$$\frac{q_f \times 12B \times 144L^2}{8} = 0.009\left[\phi_c + \frac{\phi_e}{2}\right] bd^2 f_{dy} \qquad (3.33.14a)$$

where

ϕ_e = tensile steel at the fixed support.

The terms of Equation 3.33.14a can be rearranged to yield,

$$\frac{q_f B}{b} = 0.0000416\left[\phi_c + \frac{\phi_e}{2}\right]\left(\frac{d}{L}\right)^2 f_{dy} \qquad (3.33.14b)$$

By assuming that $\phi_c = \phi_e$, Equation 3.33.14b becomes

$$\frac{q_f B}{b} = 0.0000625\,\phi_c \left(\frac{d}{L}\right)^2 f_{dy} \qquad (3.33.15)$$

The resistance of the beam or slab to diagonal tension and shear compression, assuming no web reinforcement, can be expressed as[1]

$$\frac{q_{sc} \times 12B \times 144L^2}{b} = \frac{137}{(2+\theta')} d^2 \sqrt{f'_c \phi_c} \qquad (3.33.16a)$$

Rearranging terms,

$$\frac{q_{sc} B}{b} = \frac{0.0793}{(2+\theta')} \left(\frac{d}{L}\right)^2 \sqrt{f'_c \phi_c} \qquad (3.33.16b)$$

If vertical stirrups are provided for web reinforcement, the resistance of the beam or slab to diagonal tension or shear compression is computed from the following:

$$\frac{q_{sc} B}{b} = \frac{0.0793}{(2+\theta')} \left(\frac{d}{L}\right)^2 \sqrt{f'_c \phi_c} \left[1 + 0.00002 \phi_v f_{dy}\right] \qquad (3.33.17a)$$

By rearranging terms, this becomes

$$\frac{q_{sc} B}{b} = \left[\frac{0.0793 + 0.000001586 \phi_v f_{dy}}{(2+\theta')}\right] (f'_c \phi_c)^{1/2} \left(\frac{d}{L}\right)^2 \qquad (3.33.17b)$$

The maximum shear will occur at the face of the fixed support, where the total shear $V = 5/8(q \times 12 L \times 12 B)$. The ultimate shearing resistance of the cross-section of the beam or slab can be expressed as

$$\frac{5}{8}(q_v \times 12 B \times 12 L) = V_u = 0.22 bdf'_c \qquad (3.33.18a)$$

Rearranging terms,

$$\frac{q_v B}{b} = 0.00244 f'_c \left(\frac{d}{L}\right) \qquad (3.33.18b)$$

Equations 3.33.14, 3.33.17 and 3.33.18 express the ultimate resistances q_f, q_{sc}, and q_v for a one-way reinforced concrete beam or slab,

Table 3-14

RESISTANCE FUNCTIONS FOR ONE-WAY REINFORCED CONCRETE SLABS AND BEAMS, ONE END FIXED, ONE END SIMPLY SUPPORTED

f_{dy} (psi)		44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_c$		0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_c = f'_c/k$.									
$\phi_v = 0.0$		6080	10800	8490	15090	11300	20090	17660	31390
$\phi_v = 0.5$		2930	5210	3670	6530	4410	7850	5770	10250
$\phi_v = 1.0$		1720	3060	2040	3630	2340	4150	2830	5020
$\phi_v = 1.5$		1130	2010	1300	2300	1440	2560	1670	2970

Required Depth of Beam		Resistance Function $\frac{q_f L^2}{\phi_c} = \frac{q_{sc} L^2}{\phi_c}$ (psi-sq ft)			
D(in.)	d(in.)	44,000	52,000	60,000	75,000
10	7.75	1980	2340	2700	3380
12	9.50	2980	3520	4060	5080
14	11.50	4360	5160	5950	7440
16	13.50	6010	7110	8200	10250
18	15.50	7930	9370	10810	13510
20	17.50	10110	11940	13780	17230
22	19.50	12550	14830	17110	21400
24	21.50	15250	18030	20800	26000
26	23.50	18220	21500	24900	31100
28	25.50	21500	25400	29300	36600
30	27.25	24500	29000	33400	41800
32	29.25	28200	33400	38500	48100
34	31.25	32200	38100	44000	54900
36	33.25	36500	43100	49800	62200
38	35.25	41000	48500	55900	69900
40	37.25	45800	54100	62400	78100
42	39.25	50800	60100	69300	86700
44	41.25	56200	66400	76600	95700
46	43.25	61700	73000	84200	105200
48	45.25	67600	79900	92100	115200
50	47.00	72900	86200	99400	124300
52	49.00	79200	93600	108100	135100
54	51.00	85800	101400	117100	146300
56	53.00	92700	109600	126400	158000
58	55.00	99800	118000	136100	170200
60	57.00	107200	126700	146200	182800

with one end fixed and the other end simply supported. Equating the expressions for q_f and q_{sc} yields

$$\phi_c = \left[\frac{1270 + 0.0254\phi_v f_{dy}}{f_{dy}(2+\theta')}\right]^2 f'_c \qquad \text{for } q_f = q_{sc} \quad (3.33.19)$$

The use of values of ϕ_c obtained from Equation 3.33.11 will, in theory, result in equal ultimate strengths in the flexural mode and in the diagonal tension or shear compression mode. The possibility of a failure in "pure" shear must be checked by Equation 3.33.10. As explained for the simply-supported and fixed-end cases, the value selected for ϕ_c must lie within specified maximum and minimum limits. Table 3-14 can be used to facilitate the design of slabs and beams with one fixed end and one simply-supported end. Its applicability is limited to cases where "pure" shear will not control the design.

3.33.2 Cost Studies

The anticipated widespread use of one-way reinforced concrete slabs in shelter construction warrants some detailed evaluation of their optimum costs. The total cost, C_t, of an element of a one-way slab or beam is considered as the sum of

$$C_t = C_c + C_s + C_v + C_{st} + C_f \qquad (3.33.20)$$

where

C_t = factor for composite cost per unit of structural element, (\$/sq ft)

C_c = cost factor per unit of structural element for concrete, (\$/sq ft)

C_s = cost factor per unit of structural element for reinforcing steel, (\$/sq ft)

C_v = cost factor per unit of structural element for shear steel, if required, ($/sq ft)

C_{st} = cost factor per unit of structural element for temperature reinforcement steel, if required, ($/sq ft)

C_f = cost factor per unit of structural element for form work, ($/sq ft)

To illustrate the method used in optimizing slab costs, an example is presented for the one-way reinforced slab or beam with fixed ends. The approach is a general one, and similar cost studies can be performed with other assumptions as to end restraint.

(1) Determination of Effective Slab Depth, d

The values for effective slab depth d, as used in the cost equations, are determined from the basic equation for ultimate flexural resistance of a reinforced concrete member with fixed ends. By setting $\phi_e = \phi_c$ and $B = 1/12b$ in Equation 3.33.10, the effective depth of a one-way reinforced slab with fixed ends is thus expressed as

$$d = \sqrt{\frac{1000\, q_f L^2}{\phi_c f_{dy}}} \tag{3.33.21}$$

(2) Total Cost of Concrete

The total depth of slab, D, is the sum of the effective depth d and a specified depth of concrete cover, d', over the centroid of the tension steel. Relationships between d' and d are discussed, for specific design examples, in Chapter 4 of this report. The expression for the total cost of the concrete in a reinforced concrete member then becomes

$$C_C = \frac{X_c\, D\, L\, b}{144} \tag{3.33.22}$$

(3) Total Cost of Moment Steel Reinforcement, ϕ

The term ϕ_c, which appears in the analytical equations, describes the maximum required percentage of positive moment reinforcing steel. Also, for the fixed end beams considered in this study, $\phi_c = \phi_e$. The total cost of this moment steel, however, is related to the average percentage rather than to the maximum percentage. Studies of typical layouts for the main reinforcing steel in fixed-end one-way reinforced beams and slabs, with $\theta' = 0.25$ and $\phi_c = \phi_e$, indicate the approximate relationship

$$\phi \text{ (average)} \approx \left[1.33 + \frac{0.278}{L} \quad \frac{f_{dy}}{f'_c}\right] \phi_c \ .$$

Introducing this simplification, the total cost of the moment steel in a one-way reinforced fixed-end concrete member can then be written as

$$C_S = X_s \left[1.33 + \frac{0.278}{L} \quad \frac{f_{dy}}{f'_c}\right]\left[\frac{\phi_c}{100} \times \frac{b}{12} \times \frac{d}{12} \times L\right] \qquad (3.33.23a)$$

or

$$C_S = \left[1.33 + \frac{0.278}{L} \quad \frac{f_{dy}}{f'_c}\right]\left[\frac{\phi_c bd\, L\, X_s}{14,400}\right] \qquad (3.33.23b)$$

(4) Total Cost of Diagonal Tension Reinforcement, ϕ_v

As in the analysis of the cost of moment reinforcement, it is necessary to relate the maximum value of ϕ_v (as supplied by the analytical equations) to its average value over the length of the beam or slab. Since the concrete and the stirrups are assumed to provide additive resistance to diagonal tension stresses, even for cases where $\phi_v \neq o$ it may result that no stirrups are required in regions of low shearing stresses. Studies of typical layouts for stirrup reinforcement in fixed-end beams indicate the approximate relationships

$$\phi_v \text{ (average)} \approx \left[\frac{0.000015\, \phi_v\, f_{dy}}{1 + 0.000020\, \phi_v\, f_{dy}}\right] \phi_v$$

With this simplification, Equation (3.33.25a) can be written in terms of the total cross sectional area (sq. in) of stirrups required for diagonal tension reinforcement in a one-way reinforced fixed-end beam or slab

$$A_v \text{ (total)} = \left[\frac{0.000015\, \phi_v\, f_{dy}}{1 + 0.000020\, \phi_v\, f_{dy}}\right] \left[\frac{12\, \phi_v\, L\, b}{100}\right] \qquad (3.33.25a)$$

The volume of stirrup steel is dependent upon the detailed layout of the reinforcement. For a one-way beam, assuming that typical "U" stirrups are hooked over the moment steel, the volume (cu. in) of stirrup steel is

$$V_v = \frac{1}{2} A_v \text{ (total)} \left[2(d-d') + b' + 2 \text{ hooks}\right]$$

Where b' is the width of each stirrup. This relationship can be approximated, for computing purposes, as

$$V_v = A_v \text{ (total)} \left[d + \frac{b}{2}\right]$$

With this further approximation, the total cost of stirrups in a fixed-end, one-way reinforced concrete beam is

$$C_V = \frac{X_v L\, bd\, \phi_v}{28,800} \left[\frac{0.000015\, \phi_v\, f_{dy}}{1 + 0.000020\, \phi_v\, f_{dy}}\right]\left[2 + \frac{b}{d}\right] \qquad (3.33.26)$$

For fixed-end, one-way reinforced slabs, it is visualized that the stirrups would be fabricated as a continuous frame. The required width b' between vertical portions can, for this case, be approximately related to the effective slab depth. If there are n vertical members in each diagonal tension reinforcement frame set transverse to the longitudinal bending axis, then b = 12 B and

$$V_v = A_v \text{ (total)} \left[d - d' + \left(\frac{12\,B - 2\,d'}{n-1}\right)\right].$$ Here the term $\left(\frac{12B - 2d'}{n-1}\right)$ replaces the term b' considered for the beam. Substituting $\left[-d' + \left(\frac{12\,B - 2\,d'}{n-1}\right)\right]$ for $\frac{d}{2}$, on the reasoning that spacing of individual stirrups should not exceed one-half the effective slab depth, an approximation of the total volume (cu. in) of stirrup steel in a one-way slab is

$$V_v = 3/2\ dA_v\ \text{(total)}$$

With these approximations the total cost of stirrups in a fixed-end, one-way reinforced concrete slab is

$$C_v = \frac{X_v 12BL\ \ d\,\phi_v}{800} \left[\frac{0.000015\,\phi_v\, f_{dy}}{1 + 0.000020\,\phi_v f_{dy}}\right] \qquad (3.33.27)$$

(5) Total Cost of Temperature Steel, ϕ_{te}

A normal requirement[37] for the area of temperature reinforcement steel in a one-way reinforced slab is 0.002bD. However, since the cover soil over the buried slab will reduce temperature variations, $\phi_{te} = 0.10$ is specified in this study. The total cost of the temperature steel in a one-way slab can then be expressed as

$$C_{ST} = X_s \left[\frac{\phi_{te}}{100}\right]\left[\frac{b\,D\,L}{144}\right] \qquad (3.33.28a)$$

or

$$C_{ST} = \frac{X_s\,\phi_{te}\,bDL}{14,400} \qquad (3.33.28b)$$

(6) Form Work

The total cost of the form work for a one-way reinforced slab can also be expressed as a cost equation. Forming costs, due to increased bracing requirements, are influenced by slab depth. This is primarily of concern in overhead slabs where increased slab depths require stronger form work and bracing systems. A depth factor $k'_f = X_f + 0.012D$, is therefore introduced into the equation for forming costs of overhead beams and slabs. The depth factor can alternatively be expressed as $k'_f = X_f + 0.012\, d\, (1 + \frac{d'}{d})$.

Overhead Slab $$C_F = \frac{k'_f b\ L}{12} \qquad (3.33.29a)$$

Overhead Beam $$C_F = \frac{k'_f L(b + D)}{6} \qquad (3.33.29b)$$

For ground-level construction of slabs and beam, the total cost of form work is expressed as

Ground-Level Slab $$C_F = \frac{X_f b\ L}{12} \tag{3.33.29c}$$

Ground -Level Beam $$C_F = \frac{X_f L(b + D)}{6} \tag{3.33.29d}$$

(7) Cost Factors Per Square Foot of Fixed-End One-Way Slab

The total costs given by Equations 3.33.22 to 3.33.29, inclusive, can be expressed in terms of dollar cost per square foot of slab by substituting the individual cost factors into Equation 3.33.20.

$$C_t = C_c + C_s + C_v + C_{st} + C_f \tag{3.33.20}$$

where

$$C_c = \frac{12\ C_C}{bL} = \frac{X_c\ D}{12} \tag{3.33.30a}$$

$$C_s = \frac{12\ C_S}{bL} = \frac{X_s\ \phi_c\ d}{1200}\left[1.33 + \frac{0.278}{L}\ \frac{f_{dy}}{f'_c}\right] \tag{3.33.30b}$$

$$C_v = \frac{12\ C_V}{bL} = \frac{X_v\ d\ \phi_v}{800}\left[\frac{0.000015\ \phi_v\ f_{dy}}{1 + 0.000020\ \phi_v\ f_{dy}}\right] \tag{3.33.30c}$$

$$C_{st} = \frac{12\ C_{ST}}{b\ L} = \frac{X_s\quad D}{12,000} \tag{3.33.30d}$$

Overhead Slab $$C_f = \frac{12\ C_F}{bL} = k'_f \tag{3.33.30e}$$

Ground - Level Slab $$C_f = \frac{12\ C_F}{bL} - X_f \tag{3.33.30f}$$

(8) Cost Factors Per Lineal Foot of Fixed-End One-Way Beam

The general equation for the composite cost of a unit length of beam is the same as for the fixed-end one-way slab, except that the term for the cost of temperature reinforcement is not required. With this modification, we obtain

$$C_t = C_c + C_s + C_v + C_f \qquad (3.33.31)$$

Equations 3.33.30a and 3.33.30b, after multiplying by $\frac{b}{12}$, express factors C_c and C_s for a lineal foot of beam. The factor C_v can be obtained from Equation 3.33.26, after dividing by the beam length, L. The cost of forming a lineal foot of beam is given by

$$\text{Overhead Beam} \qquad C_f = \frac{C_F}{L} = k'_f \; \frac{(b + 2D)}{12} \qquad (3.33.32a)$$

$$\text{Ground-Level Beam} \qquad C_f = \frac{C_F}{L} = X_f \left(\frac{b + 2D}{12}\right) \qquad (3.33.32b)$$

(9) Minimum-Cost Solution for Fixed-End One-Way Reinforced Concrete Slab

Equations 3.33.20 and 3.33.30 express the cost of a square foot of fixed-end one-way slab by relating unit material costs to material volumes. However, these cost equations in themselves provide no guidance to the proper combination of material and design parameters for optimum in-place cost. Such information is of prime importance, since the slab element is generally the most costly single element in a cubicle structure. When total structural costs for reinforced concrete shelters are studied, it becomes apparent that the cost of the reinforcing steel is the most influential of those material parameters which can be varied by the designer. The cost factors supplied by Equation 3.33.30 can be expressed in terms of the variable ϕ_c, through the relationship given by Equation 3.33.21. Also, the design restriction that $q_f = q_{sc}$ can be introduced by requiring that values of ϕ_c satisfy both Equation 3.33.21 and Equation 3.33.24.

The expanded cost factors so obtained, when substituted in Equation 3.33.20, result in a general expression for the unit cost of a one-way reinforced concrete slab with fixed edge support (see Section 3.34.4). Since d and D both appear in this equation, the assumption that d = 0.9 D is next introduced. Two cases are then considered (1) $\phi_v = 0$ (2) $0.50 \leq \phi_v \leq 1.50$. Obviously,

the cost factor C_v is only included in solutions where $\phi_v \neq 0$. The two cost equations are as follows.

Without Stirrups -

$$C_t = \frac{X_c D}{12} + \frac{X_s \phi_c D}{1333}\left[1.33 + \frac{0.278}{L} \frac{f_{dy}}{f'_c}\right] +$$

$$\frac{X_s D}{12,000} + X_f + 0.012 D \qquad (3.33.33)$$

With Stirrups -

$$C_t = \frac{X_c D}{12} + \frac{X_s \phi_c D}{1333}\left[1.33 + \frac{0.278}{L} \frac{f_{dy}}{f'_c}\right] + \qquad (3.33.34)$$

$$\frac{X_v \phi_v D}{888.9}\left[\frac{0.000015 \phi_v f_{dy}}{1 + 0.000020 \phi_v f_{dy}}\right] + \frac{X_s D}{12,000} + X_f + 0.012 D$$

where

$$D = \left[\frac{L}{22.326 + 0.00044651 \phi_v f_{dy}}\right]\left[q \frac{f_{dy}}{f'_c}\right]^{1/2} \qquad (3.33.35)$$

Minimum-cost solutions of Equations 3.33.33 and 3.33.34, obtained from a minimization program prepared for the IBM 7090 computer, are listed in Table 3-15. For each clear-span length of slab (ft) and equivalent loading (psi), and for each specified dynamic yield strength of reinforcing steel (psi), the Table supplies values of f'_c (psi), ϕ_c and ϕ_v (%), D (in), corresponding with minimum in-place cost C_t (\$/sq ft). In several cases, the computed cost C_t would be slightly reduced if the requirement that $\phi_v \geq 0.50$ were replaced by a requirement that $\phi_v \geq 0.15$, in conformance with conventional code requirements.[37] The values of L shown in Table 3-15 were selected on the basis of preliminary layouts for shelters of a cubicle configuration. The Table can be used to select the optimum one-way slab for a specified span and equivalent loading, if the basic assumptions of the cost equation are reasonably well satisfied. These assumptions include:

a) Slab is uniform in thickness, one-way reinforced, with fixed-edge support. The value of L, as shown in Table 3-15, is the clear-span distance between fixed supports.

b) Slab is loaded to ultimate capacity in flexure and in diagonal tension or shear compression. Pure shear does not control the design. This assumption should be checked by the designer using Equation 3.33.3.

c) θ', the ratio of negative to positive reinforcement, is equal to 0.25.

d) The percentage of all moment steel, averaged over the length of the slab, is equal to $\left[1.33 + \frac{0.278}{L} \frac{f_{dy}}{f'_c}\right] \phi_c$. This incorporates the assumption that the average diameter of main reinforcing bars is 1.00 in.

e) If web reinforcement is used, the volume of stirrup steel per square foot of slab is related to the computed maximum stirrup requirement, ϕ_v, by the relationship

$$V_v = \frac{1.5\, d\, \phi v}{100} \left[\frac{0.000015\, \phi v\, f_{dy}}{1 + 0.000020\, \phi v\, f_{dy}}\right]$$

Further, all stirrups are required to be vertical.

f) The effective depth of the slab is equal to 0.9 times its total depth. This assumption becomes questionable for very shallow or very deep slabs. For such cases, a simple correction based on the increment or decrement in concrete cover should supply a reasonable estimate of in-place cost.

g) Restrictions on the percentages of main reinforcement and of stirrup steel, and of applicable range of concrete static strengths, are as follows.

$$0.25 \leq \phi_c \leq 2.00$$

$$\phi_v = 0 \text{ or } 0.50 \leq \phi_v \leq 1.50$$

$$20000 \leq f'_c \leq 6000$$

TABLE 3-15

OPTIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END ONE WAY REINFORCED CONCRETE SLABS

($0.25 \leq \phi_c \leq 2.00$, $\phi_v = 0$ or $0.50 \leq \phi_v \leq 1.50$, f_{dy} $2000 \leq f'_c \leq 6000$, $d = 0.8\ D$)

L ft	q psi	f_{dy} psi	f'_c psi	ϕ_c %	ϕ_v %	D in	Ct $/sq ft
7.0	10.	44000.	2100.	0.68	0.	4.5	1.75
		52000.	2500.	0.57	0.	4.5	1.71
		60000.	2900.	0.50	0.	4.5	1.67
		75000.	3600.	0.40	0.	4.5	1.66
7.0	25.	44000.	4000.	1.29	0.	5.2	2.19
		52000.	5000.	1.15	0.	5.0	2.11
		60000.	5900.	1.02	0.	5.0	2.06
		75000.	6000.	0.66	0.	5.5	2.05
7.0	50.	44000.	4000.	1.29	0.	7.3	2.74
		52000.	5000.	1.15	0.	7.1	2.62
		60000.	5900.	1.02	0.	7.0	2.54
		75000.	6000.	0.66	0.	7.8	2.53
7.0	75.	44000.	4000.	1.29	0.	8.9	3.16
		52000.	5000.	1.15	0.	8.7	3.01
		60000.	5900.	1.02	0.	8.6	2.92
		75000.	5900.	0.66	0.	9.6	2.90
7.0	100.	44000.	4000.	1.29	0.	10.3	3.51
		52000.	5000.	1.14	0.	10.1	3.34
		60000.	5900.	1.02	0.	9.9	3.23
		75000.	6000.	0.66	0.	11.1	3.21
7.0	150.	44000.	4000.	1.29	0.	12.6	4.10
		52000.	5000.	1.15	0.	12.3	3.90
		60000.	5900.	1.02	0.	12.2	3.76
		75000.	6000.	0.66	0.	13.6	3.74
7.0	200.	44000.	4000.	1.29	0.	14.6	4.60
		52000.	5000.	1.15	0.	14.3	4.36
		60000.	5900.	1.02	0.	14.1	4.21
		75000.	6000.	0.66	0.	15.7	4.18
7.0	250.	44000.	4000.	1.29	0.	16.3	5.04
		52000.	5000.	1.15	0.	15.9	4.77
		60000.	5900.	1.02	0.	15.7	4.60
		75000.	6000.	0.66	0.	17.5	4.57
7.0	300.	44000.	4000.	1.29	0.	17.9	5.43
		52000.	5000.	1.15	0.	17.5	5.15
		60000.	5900.	1.02	0.	17.2	4.96
		75000.	5900.	0.66	0.	19.2	4.92
7.0	350.	44000.	4000.	1.29	0.	19.3	5.80
		52000.	5000.	1.15	0.	18.9	5.49
		60000.	5900.	1.02	0.	18.6	5.28
		75000.	5900.	0.66	0.	20.7	5.25

TABLE 3-15 (Cont'd)

OPTIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END
ONE WAY REINFORCED CONCRETE SLABS

($0.25 \leqslant \phi_c \leqslant 2.00$, $\phi_v = 0$ or $0.50 \leqslant \phi_v \leqslant 1.50$,
f_{dy} $2000 \leqslant ff'_{cc} \leqslant 6000$, $\phi_c d = 0.69 \phi_v D$)

L ft	q psi	f_{dy} psi	ff'_{cc} psi	ϕ_c %	ϕ_v %	D in	Ct $/sq ft
10.5	10.	44000.	3700.	1.18	0.	5.1	2.07
		52000.	4600.	1.06	0.	5.0	2.00
		60000.	5500.	0.95	0.	4.9	1.96
		75000.	6000.	0.66	0.	5.3	1.95
10.5	25.	44000.	3700.	1.18	0.	8.1	2.76
		52000.	4600.	1.06	0.	7.9	2.66
		60000.	5500.	0.95	0.	7.7	2.58
		75000.	5900.	0.66	0.	8.3	2.57
10.5	50.	44000.	3700.	1.18	0.	11.5	3.55
		52000.	4600.	1.06	0.	11.1	3.39
		60000.	5500.	0.95	0.	10.9	3.29
		75000.	6000.	0.66	0.	11.8	3.27
10.5	75.	44000.	3700.	1.18	0.	14.0	4.14
		52000.	4600.	1.06	0.	13.6	3.96
		60000.	5500.	0.95	0.	13.4	3.83
		75000.	6000.	0.66	0.	14.4	3.81
10.5	100.	44000.	3700.	1.18	0.	16.2	4.65
		52000.	4600.	1.06	0.	15.7	4.43
		60000.	5500.	0.95	0.	15.4	4.29
		75000.	5900.	0.66	0.	16.6	4.26
10.5	150.	44000.	3700.	1.18	0.	19.8	5.50
		52000.	4600.	1.06	0.	19.3	5.23
		60000.	5500.	0.95	0.	18.9	5.06
		75000.	6000.	0.66	0.	20.4	5.02
10.5	200.	44000.	3700.	1.18	0.	22.9	6.21
		52000.	4600.	1.06	0.	22.2	5.90
		60000.	5500.	0.95	0.	21.8	5.70
		75000.	6000.	0.66	0.	23.5	5.67
10.5	250.	44000.	3700.	1.18	0.	25.6	6.84
		52000.	4600.	1.06	0.	24.9	6.50
		60000.	5500.	0.95	0.	24.4	6.27
		75000.	6000.	0.66	0.	26.3	6.23
10.5	300.	44000.	3700.	1.18	0.	28.1	7.41
		52000.	4600.	1.06	0.	27.2	7.03
		60000.	5500.	0.95	0.	26.7	6.79
		75000.	6000.	0.66	0.	28.8	6.74
10.5	350.	44000.	3700.	1.18	0.	30.3	7.93
		52000.	4600.	1.06	0.	29.4	7.52
		60000.	5500.	0.95	0.	28.9	7.26
		75000.	5900.	0.66	0.	31.1	7.21

TABLE 3-15 (Cont'd)

OPTIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END
ONE WAY REINFORCED CONCRETE SLABS

$(0.25 \leq \phi_c \leq 2.00,\quad \phi_v = 0 \text{ or } 0.50 \leq \phi_v \leq 1.50,$
$2000 \leq f'_c \leq 6000,\ \phi_d = 0.5\ D)$

L ft	q psi	f_{dy} psi	f'_c psi	ϕ_c %	ϕ_v %	D in	Ct $/sq ft
17.5	10.	44000.	3400.	1.09	0.	8.9	2.79
		52000.	4300.	0.99	0.	8.6	2.69
		60000.	5200.	0.90	0.	8.4	2.62
		75000.	3900.	1.33	0.50	6.2	2.59
17.5	25.	44000.	3400.	1.09	0.	14.0	3.90
		52000.	4300.	0.99	0.	13.6	3.74
		60000.	5200.	0.90	0.	13.3	3.64
		75000.	3900.	1.33	0.50	9.7	3.59
17.5	50.	44000.	3400.	1.09	0.	19.9	5.15
		52000.	4300.	0.99	0.	19.2	4.93
		60000.	5200.	0.90	0.	18.7	4.78
		75000.	3900.	1.33	0.50	13.8	4.71
17.5	75.	44000.	3400.	1.09	0.	24.3	6.11
		52000.	4300.	0.99	0.	23.5	5.84
		60000.	5200.	0.90	0.	23.0	5.66
		75000.	3900.	1.33	0.50	16.9	5.57
17.5	100.	44000.	3400.	1.09	0.	28.1	6.92
		52000.	4300.	0.99	0.	27.1	6.60
		60000.	5200.	0.90	0.	26.5	6.40
		75000.	3900.	1.33	0.50	19.5	6.29
17.5	150.	44000.	3400.	1.09	0.	34.4	8.28
		52000.	4300.	0.99	0.	33.2	7.89
		60000.	5200.	0.90	0.	32.5	7.63
		75000.	3900.	1.33	0.50	23.9	7.51
17.5	200.	44000.	3400.	1.09	0.	39.7	9.42
		52000.	4300.	0.99	0.	38.4	8.97
		60000.	5200.	0.90	0.	37.5	8.68
		75000.	3900.	1.33	0.50	27.6	8.54
17.5	250.	44000.	3400.	1.09	0.	44.4	10.43
		52000.	4300.	0.99	0.	42.9	9.93
		60000.	5200.	0.90	0.	41.9	9.60
		75000.	3900.	1.33	0.50	30.8	9.44
17.5	300.	44000.	3400.	1.09	0.	48.6	11.34
		52000.	4300.	0.99	0.	47.0	10.79
		60000.	5200.	0.90	0.	45.9	10.43
		75000.	3900.	1.33	0.50	33.8	10.26
17.5	350.	44000.	3400.	1.09	0.	52.5	12.18
		52000.	4300.	0.99	0.	50.7	11.59
		60000.	5200.	0.90	0.	49.6	11.20
		75000.	3900.	1.33	0.50	36.5	11.01

TABLE 3-15 (Cont'd)

OPTIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END ONE WAY REINFORCED CONCRETE SLABS

($0.25 \leqslant \phi_c \leqslant 2.00$, $\phi_v = 0$ or $0.50 \leqslant \phi_v \leqslant 1.50$, $2000 \leqslant f'_c \leqslant 6000$, $d = 0.9$ D)

L ft	q psi	f_{dy} psi	f'_c psi	ϕ_c %	ϕ_v %	D in	Ct $/sq ft
14.0	10.	44000.	3500.	1.12	0.	7.0	2.43
		52000.	4400.	1.01	0.	6.8	2.35
		60000.	5300.	0.92	0.	6.6	2.29
		75000.	4100.	1.39	0.50	4.8	2.28
14.0	25.	44000.	3500.	1.12	0.	11.1	3.33
		52000.	4400.	1.01	0.	10.7	3.20
		60000.	5300.	0.92	0.	10.5	3.11
		75000.	4100.	1.39	0.50	7.6	3.09
14.0	50.	44000.	3500.	1.12	0.	15.7	4.35
		52000.	4400.	1.01	0.	15.1	4.16
		60000.	5300.	0.92	0.	14.8	4.04
		75000.	4100.	1.39	0.50	10.8	4.01
14.0	75.	44000.	3500.	1.12	0.	19.2	5.13
		52000.	4400.	1.01	0.	18.5	4.90
		60000.	5300.	0.92	0.	18.2	4.75
		75000.	4100.	1.39	0.50	13.2	4.72
14.0	100.	44000.	3500.	1.12	0.	22.1	5.79
		52000.	4400.	1.01	0.	21.4	5.52
		60000.	5300.	0.92	0.	21.0	5.34
		75000.	4100.	1.39	0.50	15.2	5.31
14.0	150.	44000.	3500.	1.12	0.	27.1	6.89
		52000.	4400.	1.02	0.	26.2	6.56
		60000.	5300.	0.92	0.	25.7	6.35
		75000.	4100.	1.39	0.50	18.7	6.31
14.0	200.	44000.	3500.	1.12	0.	31.3	7.82
		52000.	4400.	1.01	0.	30.3	7.44
		60000.	5300.	0.92	0.	29.6	7.19
		75000.	4100.	1.39	0.50	21.6	7.14
14.0	250.	44000.	3500.	1.12	0.	35.0	8.64
		52000.	4400.	1.01	0.	33.9	8.21
		60000.	5300.	0.92	0.	33.1	7.94
		75000.	4100.	1.39	0.50	24.1	7.88
14.0	300.	44000.	3500.	1.12	0.	38.3	9.38
		52000.	4400.	1.01	0.	37.1	8.91
		60000.	5300.	0.92	0.	36.3	8.61
		75000.	4100.	1.39	0.50	26.4	8.55
14.0	350.	44000.	3500.	1.12	0.	41.4	10.06
		52000.	4400.	1.01	0.	40.1	9.56
		60000.	5300.	0.92	0.	39.2	9.23
		75000.	4100.	1.39	0.50	28.5	9.17

TABLE 3-15 (Cont'd)

OPTIMUN IN-PLACE COSTS FOR OVERHEAD FIXED-END ONE WAY REINFORCED CONCRETE SLABS

($0.25 \leqslant \phi_c \leqslant 2.00$, $\phi_v = 0$ or $0.50 \leqslant \phi_v \leqslant 1.50$, $2000 \leqslant f'_c \leqslant 6000$, d = 0.9 D)

L ft	q psi	f_{dy} psi	f'_c psi	ϕ_c %	ϕ_v %	D in	Ct $/sq ft
21.0	10.	44000.	3300.	1.07	0.	10.8	3.15
		52000.	4200.	0.97	0.	10.4	3.03
		60000.	5100.	0.88	0.	10.1	2.96
		75000.	3800.	1.29	0.50	7.5	2.90
21.0	25.	44000.	3300.	1.07	0.	17.0	4.47
		52000.	4200.	0.97	0.	16.4	4.28
		60000.	5100.	0.88	0.	16.0	4.16
		75000.	3800.	1.29	0.50	11.9	4.08
21.0	50.	44000.	3300.	1.07	0.	24.1	5.95
		52000.	4200.	0.97	0.	23.2	5.69
		60000.	5100.	0.88	0.	22.7	5.52
		75000.	3800.	1.29	0.50	16.8	5.40
21.0	75.	44000.	3300.	1.07	0.	29.5	7.09
		52000.	4200.	0.97	0.	28.4	6.77
		60000.	5100.	0.88	0.	27.8	6.57
		75000.	3800.	1.29	0.50	20.6	6.42
21.0	100.	44000.	3300.	1.07	0.	34.1	8.05
		52000.	4200.	0.97	0.	32.8	7.69
		60000.	5100.	0.88	0.	32.1	7.45
		75000.	3800.	1.29	0.50	23.8	7.28
21.0	150.	44000.	3300.	1.07	0.	41.7	9.67
		52000.	4200.	0.97	0.	40.2	9.22
		60000.	5100.	0.88	0.	39.3	8.92
		75000.	3800.	1.29	0.50	29.1	8.71
21.0	200.	44000.	3300.	1.07	0.	48.2	11.03
		52000.	4200.	0.97	0.	46.4	10.51
		60000.	5100.	0.88	0.	45.3	10.17
		75000.	3800.	1.29	0.50	33.6	9.92
21.0	250.	44000.	3300.	1.07	0.	53.8	12.22
		52000.	4200.	0.97	0.	51.9	11.64
		60000.	5100.	0.88	0.	50.7	11.26
		75000.	3800.	1.29	0.50	37.6	10.99
21.0	300.	44000.	3300.	1.07	0.	59.0	13.31
		52000.	4200.	0.97	0.	56.9	12.67
		60000.	5100.	0.88	0.	55.5	12.25
		75000.	3800.	1.29	0.50	41.2	11.96
21.0	350.	44000.	3300.	1.07	0.	63.7	14.30
		52000.	4200.	0.97	0.	61.4	13.61
		60000.	5100.	0.88	0.	60.0	13.17
		75000.	3800.	1.29	0.50	44.5	12.84

TABLE 3-15 (Cont'd)

OPTIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END ONE WAY REINFORCED CONCRETE SLABS

($0.25 \leq \phi_c \leq 2.00$, $\phi_v = 0$ or $0.50 \leq \phi_v \leq 1.50$, $2000 \leq f'_c \leq 6000$, $d = 0.9\ D$)

L ft	q psi	f_{dy} psi	f'_c psi	ϕ_c %	ϕ_v %	D in	Ct $/sq ft
28.0	10.	44000.	3200.	1.04	0.	14.6	3.87
		52000.	4100.	0.95	0.	14.0	3.72
		60000.	3000.	1.34	0.50	11.0	3.62
		75000.	3600.	1.23	0.50	10.2	3.52
28.0	25.	44000.	3200.	1.04	0.	23.0	5.60
		52000.	4100.	0.95	0.	22.1	5.37
		60000.	3000.	1.34	0.50	17.4	5.21
		75000.	3600.	1.23	0.50	16.2	5.06
28.0	50.	44000.	3200.	1.04	0.	32.5	7.56
		52000.	4100.	0.95	0.	31.3	7.22
		60000.	3000.	1.34	0.50	24.6	7.00
		75000.	3600.	1.23	0.50	22.9	6.79
28.0	75.	44000.	3200.	1.04	0.	39.8	9.06
		52000.	4100.	0.95	0.	38.3	8.65
		60000.	3000.	1.34	0.50	30.1	8.37
		75000.	3600.	1.23	0.50	28.0	8.12
28.0	100.	44000.	3200.	1.04	0.	46.0	10.32
		52000.	4100.	0.95	0.	44.3	9.85
		60000.	3000.	1.34	0.50	34.8	9.53
		75000.	3600.	1.23	0.50	32.3	9.24
28.0	150.	44000.	3200.	1.04	0.	56.4	12.44
		52000.	4100.	0.95	0.	54.2	11.87
		60000.	3000.	1.34	0.50	42.6	11.48
		75000.	3600.	1.23	0.50	39.6	11.11
28.0	200.	44000.	3200.	1.04	0.	65.1	14.23
		52000.	4100.	0.95	0.	62.6	13.57
		60000.	3000.	1.34	0.50	49.2	13.12
		75000.	3600.	1.23	0.50	45.7	12.70
28.0	250.	44000.	3200.	1.04	0.	72.8	15.81
		52000.	4100.	0.95	0.	70.0	15.07
		60000.	3000.	1.34	0.50	55.0	14.56
		75000.	3600.	1.23	0.50	51.1	14.09
28.0	300.	44000.	3200.	1.04	0.	79.7	17.23
		52000.	4100.	0.95	0.	76.7	16.42
		60000.	3000.	1.33	0.50	60.2	15.87
		75000.	3600.	1.23	0.50	56.0	15.35
28.0	350.	44000.	3200.	1.04	0.	86.1	18.54
		52000.	4100.	0.95	0.	82.6	17.67
		60000.	3000.	1.34	0.50	65.0	17.07
		75000.	3600.	1.23	0.50	60.5	16.51

3.34 Two-Way Reinforced Slabs

3.34.1 Introduction

Two systems of two-way slab reinforcement are examined and will be discussed separately. Following the definitions proposed in Reference 39, the term "isotropic reinforcement" will refer to a square mesh of equal-sized bars extending in both directions of the slab. This are considered as positive reinforcement when placed at the bottom of the slab and as negative reinforcement when placed at its top. However, the amount of steel need not be the same in the top and bottom square meshes. The term "orthotropic reinforcement" will refer to the placing of unequal amounts of reinforcement in two directions at right angles. The use of a reduced quantity of reinforcement in the long-span direction, which is the basis of orthotropic slab design, is consistent with the calculated distribution of moment in a rectangular two-way reinforced slab.

The ultimate flexural resistance of both isotropic and orthotropic slabs will be computed by yield line theory, without including any additional resistance which may develop as a result of membrane action(3). The resistance of an isotropic two-way slab to diagonal tension and shear compression stresses will be calculated from Equation 5B - 10 of Reference 2. This same equation, with appropriate modifications, will also be applied to the orthotropic two-way slab. The resistance of two-way slabs to "pure" shear will similarly be calculated from the equations of Reference 2. As with the one-way slabs, limitations will be placed on the permissible ratios of reinforcement steel to cross-sectional area of the slab(1).

3.34.2 Isotropic Reinforcement

(a) Simply-Supported Two-Way Reinforced Isotropic Slab

The maximum flexural stress will occur in the center of the short span. The general expression for the dynamic flexural resistance of a one-inch strip of two-way reinforced isotropic slab, as adapted from Reference 3, is as follows:

$$q_f = 0.000750(\phi_{Sc} + \phi_{Se})\, f_{dy} \left(\frac{d}{L_S}\right)^2 \left[\alpha\left(\frac{\phi_{Lc} + \phi_{Le}}{\phi_{Sc} + \phi_{Se}}\right) + \left(\frac{2 - \alpha}{3 - 2\alpha}\right)\right]. \qquad (3.34.1)$$

where

ϕ_{Sc} = effective tensile steel percentage at mid-span in short direction of two-way slab

ϕ_{Se} = effective tensile steel percentage at supports in short direction of two-way slab

d = effective depth of slab, (in.)

L_S = length of slab in short direction, (ft)

α = ratio of short to long spans of a two-way slab. Only values of $\alpha \geq 0.5$ will be considered.

ϕ_{Lc} = effective tensile steel percentage at mid-span in long direction of two-way slab

ϕ_{Le} = effective tensile steel percentage at supports in long direction of two-way slab

For an isotropic slab, $\phi_{Lc} = \phi_{Sc}$ and $\phi_{Le} = \phi_{Se}$. Thus, Equation 3.34.1 becomes

$$q_f = 0.00150(\phi_{Sc} + \phi_{Se})\, f_{dy} \left(\frac{d}{L_S}\right)^2 \left[\frac{1 + \alpha - \alpha^2}{3 - 2\alpha}\right] \qquad (3.34.2)$$

For the simply-supported slab, $\phi_{Se} = 0$ and Equation 3.34.2 reduces to

$$q_f = 0.00150\,\phi_{Sc}\, f_{dy} \left(\frac{d}{L_S}\right)^2 \left[\frac{1 + \alpha - \alpha^2}{3 - 2\alpha}\right] \qquad (3.34.3)$$

The resistance of a one-inch width of two-way reinforced isotropic slab to diagonal tension or shear compression, assuming simply-supported edges and no web reinforcement, can be expressed as[2],

$$q_{sc} \times 144 L_S^2 = \left(\frac{208}{2+\theta'}\right)\left(\frac{1+\alpha}{3}\right) d^2 \sqrt{f'_c \phi_{Sc}} \qquad (3.34.4a)$$

Here θ', in analogous form to that described for the one-way reinforced slab, is the ratio of negative reinforcement to positive reinforcement. A minimum value of $\theta' = 0.25$ is specified for two-way reinforced slabs.

The expression for q_{sc} may be further reduced.

$$q_{sc} = \frac{1.446}{2+\theta'}\left(\frac{d}{L_S}\right)^2 (f'_c \phi_{Sc})^{1/2} \left(\frac{1+\alpha}{3}\right) \qquad (3.34.4b)$$

If web reinforcement is supplied in the form of vertical stirrups, Equation 3.34.4b becomes

$$q_{sc}\, 144 L_S^2 = \left(\frac{208}{2+\theta'}\right)\left(\frac{1+\alpha}{3}\right) d^2 \sqrt{f'_c \phi_{Sc}} \left[1 + 0.00002 \phi_v f_{dy}\right] \qquad (3.34.5a)$$

Rearranging terms, this expression reduces to

$$q_{sc} = \left[\frac{0.482 + 0.00000964 \phi_v f_{dy}}{(2+\theta')}\right] (f'_c \phi_{Sc})^{1/2} \left(\frac{d}{L_S}\right)^2 (1+\alpha) \qquad (3.34.5b)$$

The maximum shear occurs at the simply-supported edge of the short span. The resistance of a one-inch width of two-way reinforced isotropic slab to "pure" shear is expressed as[2],

$$q_v = 0.0245\,(1+\alpha)\,f'_c \left(\frac{d}{L_S}\right) \qquad (3.34.6)$$

Equations 3.34.4 to 3.34.6, inclusive, express the ultimate resistances of a one-inch width of two-way reinforced isotropic slab, simply supported, to the three failure modes analyzed herein. As with the one-way slab, the expressions for q_f and q_{sc} can be equated to find values of ϕ_{Sc} which, in theory, will provide equal ultimate resistances in flexure and in diagonal tension or shear compression. This procedure yields the following equation.

$$\phi_{Sc} = \left[\frac{1450 + 0.0289\,\phi_v f_{dy}}{f_{dy}\,(2+0')}\right]^2 \left[\frac{3+\alpha-2\alpha^2}{4.5\,(1+\alpha-\alpha^2)}\right]^2 f'_c \qquad (3.34.7)$$

Or, expressing this balanced value of ϕ_{Sc} in terms of the balanced ϕ_c for a one-way reinforced slab with the same support conditions and material properties, as expressed by Equation 3.33.8b, the relation between ϕ_{Sc} and ϕ_c is as follows.

$$\phi_{Sc} = \phi_c \left[\frac{3+\alpha-2\alpha^2}{4.5\,(1+\alpha-\alpha^2)}\right]^2 \qquad (3.34.8)$$

Values of ϕ_{Sc} obtained from Equation 3.34.7 must be checked by use of Equation 3.34.6 or Figure 3-4 to ensure that the slab resistance to "pure" shear will not control the design. Also, ϕ_{Sc} must lie within acceptable minimum and maximum values, as discussed for the one-way reinforced slab.

Tables 3-16 to 3-21 contain resistance functions calculated for α = 1.0, 0.9, 0.8, 0.7, 0.6 and 0.5. Isotropic two-way reinforced slabs with simply-supported edges can be designed with the aid of these resistance functions, for cases where "pure" shear does not control, following the procedures described for one-way reinforced slabs.

Table 3-16

RESISTANCE FUNCTIONS FOR TWO-WAY ISOTROPIC REINFORCED CONCRETE SLABS, SIMPLY SUPPORTED (α = 1.0)

f_{dy} (psi)		44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$		0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.									
$\phi_v = 0.0$		23600	42000	33000	58600	43900	78000	68600	121900
$\phi_v = 0.5$		11410	20300	14300	25400	17190	30600	22500	39900
$\phi_v = 1.0$		6700	11910	7950	14130	9100	16180	11020	19580
$\phi_v = 1.5$		4400	7830	5050	8980	5620	9990	6520	11590
Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)							
D(in.)	d(in.)								
10	7.75	3960		4680		5410		6760	
12	9.50	5960		7040		8120		10150	
14	11.50	8730		10320		11900		14880	
16	13.50	12030		14220		16400		20500	
18	15.50	15860		18740		21600		27000	
20	17.50	20200		23900		27600		34500	
22	19.50	25100		29700		34200		42800	
24	21.50	30500		36100		41600		52000	
26	23.50	36400		43100		49700		62100	
28	25.50	42900		50700		58500		73200	
30	27.25	49000		57900		66800		83500	
32	29.25	56500		66700		77000		96300	
34	31.25	64500		76200		87900		109900	
36	33.25	73000		86200		99500		124400	
38	35.25	82000		96900		111800		139800	
40	37.25	91600		108200		124900		156100	
42	39.25	101700		120200		138700		173300	
44	41.25	112300		132700		153100		191400	
46	43.25	123500		145900		168400		210000	
48	45.25	135100		159700		184300		230000	
50	47.00	145800		172300		198800		249000	
52	49.00	158500		187300		216000		270000	
54	51.00	171700		203000		234000		293000	
56	53.00	185400		219000		253000		316000	
58	55.00	199600		236000		272000		340000	
60	57.00	214000		253000		292000		366000	

Table 3-17

RESISTANCE FUNCTIONS FOR TWO-WAY ISOTROPIC REINFORCED CONCRETE SLABS; SIMPLY SUPPORTED (α = 0.9)

f_{dy} (psi)		44,000		52,000		60,000		75,000	
$Q' = \phi'/\phi_{Sc}$		0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.									
ϕ_v = 0.0		21580	38350	30130	53570	40120	71320	62680	111430
ϕ_v = 0.5		10430	18540	13070	23240	15710	27930	20530	36490
ϕ_v = 1.0		6120	10880	7260	12920	8320	14790	10070	17900
ϕ_v = 1.5		4020	7150	4620	8210	5140	9140	5960	10600
Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)							
D(in.)	d(in.)								
10	7.75	3620		4270		4930		6160	
12	9.50	5430		6420		7410		9260	
14	11.50	7960		9410		10860		13570	
16	13.50	10970		12970		14960		18700	
18	15.50	14460		17090		19720		24700	
20	17.50	18430		21800		25100		31400	
22	19.50	22900		27100		31200		39000	
24	21.50	27800		32900		37900		47400	
26	23.50	33200		39300		45300		56700	
28	25.50	39100		46300		53400		66700	
30	27.25	44700		52800		61000		76200	
32	29.25	51500		60900		70200		87800	
34	31.25	58800		69500		80200		100200	
36	33.25	66500		78600		90700		113400	
38	35.25	74800		88400		102000		127500	
40	37.25	83500		98700		113900		142400	
42	39.25	92700		109600		126500		158100	
44	41.25	102400		121000		139700		174600	
46	43.25	112600		133100		153500		191900	
48	45.25	123300		145700		168100		210000	
50	47.00	133000		157100		181300		227000	
52	49.00	144500		170800		197100		246000	
54	51.00	156600		185000		213000		267000	
56	53.00	169100		199800		231000		288000	
58	55.00	182100		215000		248000		310000	
60	57.00	195600		231000		267000		333000	

Table 3-18

RESISTANCE FUNCTIONS FOR TWO-WAY ISOTROPIC REINFORCED CONCRETE SLABS, SIMPLY SUPPORTED (α = 0.8)

f_{dy} (psi)		44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$		0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.									
ϕ_v = 0.0		20000	35560	27940	49670	37190	66120	58120	103320
ϕ_v = 0.5		9670	17180	12120	21550	14570	25900	19030	33830
ϕ_v = 1.0		5680	10090	6740	11980	7720	13710	9340	16600
ϕ_v = 1.5		3730	6630	4280	7610	4770	8470	5530	9830
Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)							
D(in.)	d(in.)								
10	7.75	3250		3840		4430		5540	
12	9.50	4880		5770		6660		8320	
14	11.50	7150		8450		9750		12190	
16	13.50	9860		11650		13440		16800	
18	15.50	12990		15360		17720		22100	
20	17.50	16560		19570		22600		28200	
22	19.50	20600		24300		28000		35100	
24	21.50	25000		29500		34100		42600	
26	23.50	29900		35300		40700		50900	
28	25.50	35200		41600		48000		59900	
30	27.25	40200		47500		54800		68500	
32	29.25	46300		54700		63100		78900	
34	31.25	52800		62400		72000		90000	
36	33.25	59800		70700		81500		101900	
38	35.25	67200		79400		91600		114500	
40	37.25	75000		88700		102300		127900	
42	39.25	83300		98500		113600		142000	
44	41.25	92000		108800		125500		156900	
46	43.25	101200		119600		138000		172400	
48	45.25	110700		130900		151000		188800	
50	47.00	119500		141200		162900		204000	
52	49.00	129900		153500		177100		221000	
54	51.00	140700		166200		191800		240000	
56	53.00	151900		179500		207000		259000	
58	55.00	163600		193300		223000		279000	
60	57.00	175700		208000		240000		300000	

Table 3-19

RESISTANCE FUNCTIONS FOR TWO-WAY ISOTROPIC REINFORCED CONCRETE SLABS, SIMPLY SUPPORTED (α = 0.7)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.								
$\phi_v = 0.0$	18680	33210	26090	46380	34730	61750	54270	96480
$\phi_v = 0.5$	9030	16050	11320	20120	13600	24180	17780	31600
$\phi_v = 1.0$	5300	9420	6290	11180	7200	12810	8720	15500
$\phi_v = 1.5$	3480	6190	4000	7110	4450	7910	5160	9180

Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)			
D(in.)	d(in.)				
10	7.75	2990	3540	4080	5100
12	9.50	4490	5310	6130	7660
14	11.50	6590	7780	8980	11230
16	13.50	9080	10730	12380	15470
18	15.50	11970	14140	16320	20400
20	17.50	15250	18030	20800	26000
22	19.50	18940	22400	25800	32300
24	21.50	23000	27200	31400	39200
26	23.50	27500	32500	37500	46900
28	25.50	32400	38300	44200	55200
30	27.25	37000	43700	50400	63000
32	29.25	42600	50400	58100	72600
34	31.25	48600	57500	66300	82900
36	33.25	55100	65100	75100	93900
38	35.25	61900	73100	84400	105500
40	37.25	69100	81700	94200	117800
42	39.25	76700	90700	104600	130800
44	41.25	84700	100200	115600	144500
46	43.25	93200	110100	127000	158800
48	45.25	102000	120500	139100	173800
50	47.00	110000	130000	150000	187500
52	49.00	119600	141300	163100	204000
54	51.00	129500	153100	176700	221000
56	53.00	139900	165300	190800	238000
58	55.00	150700	178100	205000	257000
60	57.00	161800	191200	221000	276000

Table 3-20

RESISTANCE FUNCTIONS FOR TWO-WAY ISOTROPIC REINFORCED CONCRETE SLABS, SIMPLY SUPPORTED ($\alpha = 0.6$)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
	Read value of k. Compute $\phi_{Sc} = f'_c/k$.							
$\phi_v = 0.0$	17500	31110	24440	43450	32540	57850	50840	90390
$\phi_v = 0.5$	8460	15030	10600	18850	12740	22660	16650	29600
$\phi_v = 1.0$	4970	8830	5890	10480	6750	12000	8170	14520
$\phi_v = 1.5$	3260	5800	3750	6660	4170	7410	4840	8600

Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)			
D(in.)	d(in.)	44,000	52,000	60,000	75,000
10	7.75	2610	3080	3550	4440
12	9.50	3920	4630	5340	6670
14	11.50	5740	6780	7820	9780
16	13.50	7910	9350	10780	13480
18	15.50	10420	12320	14210	17770
20	17.50	13290	15700	18120	22600
22	19.50	16500	19500	22500	28100
24	21.50	20100	23700	27300	34200
26	23.50	24000	28300	32700	40800
28	25.50	28200	33300	38500	48100
30	27.25	32200	38100	43900	54900
32	29.25	37100	43900	50600	63300
34	31.25	42400	50100	57800	72200
36	33.25	48000	56700	65400	81800
38	35.25	53900	63700	73500	91900
40	37.25	60200	71200	82100	102600
42	39.25	66800	79000	91100	113900
44	41.25	73800	87300	100700	125800
46	43.25	81200	95900	110700	138300
48	45.25	88800	105000	121100	151400
50	47.00	95800	113300	130700	163400
52	49.00	104200	123100	142100	177600
54	51.00	112900	133400	153900	192400
56	53.00	121900	144000	166200	208000
58	55.00	131300	155100	179000	224000
60	57.00	141000	166600	192200	240000

Table 3-21

RESISTANCE FUNCTIONS FOR TWO-WAY ISOTROPIC REINFORCED CONCRETE SLABS, SIMPLY SUPPORTED (α = 0.5)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.								
ϕ_v = 0.0	16390	29140	22890	40700	30480	54180	47620	84660
ϕ_v = 0.5	7920	14080	9930	17660	11940	21220	15600	27720
ϕ_v = 1.0	4650	8270	5520	9810	6320	11240	7650	13600
ϕ_v = 1.5	3060	5440	3510	6240	3910	6940	4530	8050

Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)			
D(in.)	d(in.)	44,000	52,000	60,000	75,000
10	7.75	2330	2750	3180	3970
12	9.50	3500	4140	4780	5970
14	11.50	5130	6070	7000	8750
16	13.50	7070	8360	9640	12060
18	15.50	9320	11020	12710	15890
20	17.50	11880	14040	16210	20300
22	19.50	14760	17440	20100	25200
24	21.50	17940	21200	24500	30600
26	23.50	21400	25300	29200	36500
28	25.50	25200	29800	34400	43000
30	27.25	28800	34100	39300	49100
32	29.25	33200	39200	45300	56600
34	31.25	37900	44800	51700	64600
36	33.25	42900	50700	58500	73100
38	35.25	48200	57000	65800	82200
40	37.25	53800	63600	73400	91800
42	39.25	59800	70700	81500	101900
44	41.25	66000	78000	90000	112600
46	43.25	72600	85800	99000	123700
48	45.25	79500	93900	108400	135400
50	47.00	85700	101300	116900	146100
52	49.00	93200	110100	127100	158800
54	51.00	100900	119300	137600	172000
56	53.00	109000	128800	148600	185800
58	55.00	117400	138700	160100	200000
60	57.00	126100	149000	171900	215000

(b) Fixed-Edge Two-Way Reinforced Isotropic Slab

The maximum flexural stress will occur at the center and at the supported edges of the short span. Equation 3.34.2 expresses the flexural resistance of an isotropic slab. If it is assumed that equal amounts of reinforcement are provided in the top and bottom of the slab, ($\phi_{Sc} = \phi_{Se}$), Equation 3.34.2 as applied to a one-inch width of fixed-edge slab will then become

$$q_f = 0.0030\,\phi_{Sc}\left(\frac{d}{L_S}\right)^2 f_{dy}\left(\frac{1+\alpha-\alpha^2}{3-2\alpha}\right) \qquad (3.34.9)$$

The resistance of a one-inch width of two-way reinforced isotropic slab to diagonal tension or shear compression, assuming fixed edges and no web reinforcement, can be expressed as follows[2].

$$q_{sc} = \frac{1.175}{2+\theta'}\left(\frac{d}{L_S}\right)^2 (f'_c\,\phi_{Sc})^{1/2}(1+\alpha) \qquad (3.34.10a)$$

If web reinforcement is supplied in the form of vertical stirrups, Equation 3.34.10a becomes

$$q_{sc} = \left[\frac{1.175+0.00002344\,\phi_v f_{dy}}{(2+\theta')}\right]\left[(f'_c\,\phi_{Sc})^{\frac{1}{2}}\left(\frac{d}{L_S}\right)^2(1+\alpha)\right] \qquad (3.34.10b)$$

The resistance of the fixed-edge isotropic slab to "pure" shear is the same as for the simply-supported slab.

$$q_v = 0.0245\,(1+\alpha)\,f'_c\left(\frac{d}{L_S}\right) \qquad (3.34.6)$$

Equations 3.33.9, 3.33.10 and 3.34.6 express the ultimate resistances of a one-inch width of two-way reinforced isotropic slab, fixed-edge support, to the three postulated failure modes. The expressions for q_f and q_{sc} may be solved, as for the simply-supported case, to obtain values of ϕ_{Sc} which theoretically correspond to equal ultimate resistances in flexure and in diagonal tension or shear compression. The resulting equation for ϕ_{Sc} is

$$\phi_{Sc} = \left[\frac{1765 + 0.0353\,\phi_v f_{dy}}{f_{dy}(2+\Theta')}\right]^2 f'_c \left[\frac{3+\alpha-2\alpha^2}{4.5(1+\alpha-\alpha^2)}\right]^2 \qquad (3.34.11)$$

This equation for balanced ϕ_{Sc} may also be expressed in terms of the balanced ϕ_c for a one-way reinforced slab with the same support conditions and material properties. The resulting equation is identical with that previously derived for simple-support conditions.

$$\phi_{Sc} = \phi_c \left[\frac{3+\alpha-2\alpha^2}{4.5(1+\alpha-\alpha^2)}\right]^2 \qquad (3.34.8)$$

Values of ϕ_{Sc} obtained from Equation 3.34.11 must be checked by use of Equation 3.34.6 or Figure 3-4 to ensure that the resistance of the slab to "pure" shear will not control the design. As previously explained, ϕ_{Sc} must also lie within acceptable minimum and maximum limits.

Tables 3-22 to 3-27 contain resistance functions calculated for α = 1.0, 0.9, 0.8, 0.7, 0.6 and 0.5. Isotropic two-way slabs with fixed-edge support can be designed with the aid of these resistance functions, for cases where "pure" shear does not control, following the procedures described for one-way reinforced slabs.

3.34.3 Orthotropic Reinforcement

After introducing an affine transformation from L_S to L'_S, the orthotropic slab will be treated as an isotropic slab[39]. In this way,

Table 3-22

RESISTANCE FUNCTIONS FOR TWO-WAY ISOTROPIC REINFORCED CONCRETE SLABS, FIXED EDGE SUPPORTS (α = 1.0)

f_{dy} (psi)		44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$		0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
		Read value of k. Compute $\phi_{Sc} = f'_c/k$.							
ϕ_v = 0.0		15930	28310	22240	39550	29620	52650	46280	82270
ϕ_v = 0.5		7680	13650	9630	17120	11570	20570	15110	26860
ϕ_v = 1.0		4510	8010	5350	9500	6120	10880	7410	13160
ϕ_v = 1.5		2960	5260	3390	6030	3780	6710	4380	7790
Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)							
D(in.)	d(in.)								
10	7.75	7930		9370		10810		13510	
12	9.50	11910		14080		16240		20300	
14	11.50	17460		20600		23800		29800	
16	13.50	24100		28400		32800		41000	
18	15.50	31700		37500		43200		54100	
20	17.50	40400		47800		55100		68900	
22	19.50	50200		59300		68400		85600	
24	21.50	61000		72100		83200		104000	
26	23.50	72900		86200		99400		124300	
28	25.50	85800		101400		117000		146300	
30	27.25	98000		115800		133700		167100	
32	29.25	112900		133500		154000		192500	
34	31.25	128900		152300		175800		220000	
36	33.25	145900		172500		199000		249000	
38	35.25	164000		193800		224000		280000	
40	37.25	183200		216000		250000		312000	
42	39.25	203000		240000		277000		347000	
44	41.25	225000		265000		306000		383000	
46	43.25	247000		292000		337000		421000	
48	45.25	270000		319000		369000		461000	
50	47.00	292000		345000		398000		497000	
52	49.00	317000		375000		432000		540000	
54	51.00	343000		406000		468000		585000	
56	53.00	371000		438000		506000		632000	
58	55.00	399000		472000		544000		681000	
60	57.00	429000		507000		585000		731000	

Table 3-23

RESISTANCE FUNCTIONS FOR TWO-WAY ISOTROPIC REINFORCED CONCRETE SLABS, FIXED EDGE SUPPORTS (α = 0.9)

f_{dy} (psi)		44,000		52,000		60,000		75,000	
$Q' = \phi'/\phi_{Sc}$		0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.									
ϕ_v = 0.0		14560	25880	20340	36150	27070	48130	42300	75210
ϕ_v = 0.5		7020	12480	8800	15650	10570	18800	13810	24560
ϕ_v = 1.0		4120	7320	4890	8690	5600	9940	6770	12030
ϕ_v = 1.5		2700	4810	3100	5520	3450	6140	4000	7120
Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)							
D(in.)	d(in.)								
10	7.75	7230		8550		9860		12330	
12	9.50	10870		12840		14820		18520	
14	11.50	15920		18820		21700		27100	
16	13.50	21900		25900		29900		37400	
18	15.50	28900		34200		39400		49300	
20	17.50	36900		43600		50300		62800	
22	19.50	45800		54100		62400		78000	
24	21.50	55600		65800		75900		94900	
26	23.50	66500		78600		90700		113300	
28	25.50	78300		92500		106700		133400	
30	27.25	89400		105700		121900		152400	
32	29.25	103000		121700		140500		175600	
34	31.25	117600		138900		160300		200000	
36	33.25	133100		157300		181500		227000	
38	35.25	149600		176800		204000		255000	
40	37.25	167000		197400		228000		285000	
42	39.25	185500		219000		253000		316000	
44	41.25	205000		242000		279000		349000	
46	43.25	225000		266000		307000		384000	
48	45.25	247000		291000		336000		420000	
50	47.00	266000		314000		363000		453000	
52	49.00	289000		342000		394000		493000	
54	51.00	313000		370000		427000		534000	
56	53.00	338000		400000		461000		576000	
58	55.00	364000		430000		497000		621000	
60	57.00	391000		462000		533000		667000	

Table 3-24

RESISTANCE FUNCTIONS FOR TWO-WAY ISOTROPIC REINFORCED CONCRETE SLABS, FIXED EDGE SUPPORTS (α = 0.8)

f_{dy} (psi)		44,000		52,000		60,000		75,000	
$Q' = \phi'/\phi_{Sc}$		0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.									
ϕ_v = 0.0		13500	24000	18850	33520	25100	44630	39220	69730
ϕ_v = 0.5		6510	11570	8160	14510	9800	17430	12810	22770
ϕ_v = 1.0		3820	6790	4530	8050	5190	9220	6280	11160
ϕ_v = 1.5		2500	4460	2870	5110	3200	5690	3710	6600
Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)							
D(in.)	d(in.)								
10	7.75	6500		7680		8860		11070	
12	9.50	9760		11540		13310		16640	
14	11.50	14310		16910		19510		24400	
16	13.50	19710		23300		26900		33600	
18	15.50	26000		30700		35400		44300	
20	17.50	33100		39100		45200		56500	
22	19.50	41100		48600		56100		70100	
24	21.50	50000		59100		68200		85200	
26	23.50	59700		70600		81500		101800	
28	25.50	70300		83100		95900		119900	
30	27.25	80300		94900		109500		136900	
32	29.25	92500		109400		126200		157700	
34	31.25	105600		124800		144000		180100	
36	33.25	119600		141300		163100		204000	
38	35.25	134400		158800		183300		229000	
40	37.25	150100		177400		205000		256000	
42	39.25	166600		196900		227000		284000	
44	41.25	184100		218000		251000		314000	
46	43.25	202000		239000		276000		345000	
48	45.25	221000		262000		302000		378000	
50	47.00	239000		282000		326000		407000	
52	49.00	260000		307000		354000		443000	
54	51.00	281000		332000		384000		480000	
56	53.00	304000		359000		414000		518000	
58	55.00	327000		387000		446000		558000	
60	57.00	351000		415000		479000		599000	

Table 3-25

RESISTANCE FUNCTIONS FOR TWO-WAY ISOTROPIC REINFORCED CONCRETE SLABS, FIXED EDGE SUPPORTS (α = 0.7)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.								
ϕ_v = 0.0	12610	22410	17610	31300	23440	41670	36630	65120
ϕ_v = 0.5	6080	10810	7620	13550	9160	16280	11960	21260
ϕ_v = 1.0	3570	6340	4230	7520	4840	8610	5862	10420
ϕ_v = 1.5	2340	4160	2680	4780	2990	5310	3470	6160

Required Depth of Beam D(in.)	Required Depth of Beam d(in.)	Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft) 44,000	52,000	60,000	75,000
10	7.75	5980	7070	8160	10200
12	9.50	8990	10620	12260	15320
14	11.50	13170	15570	17960	22500
16	13.50	18150	21500	24800	30900
18	15.50	23900	28300	32600	40800
20	17.50	30500	36100	41600	52000
22	19.50	37900	44800	51700	64600
24	21.50	46000	54400	62800	78500
26	23.50	55000	65000	75000	93800
28	25.50	64800	76500	88300	110400
30	27.25	74000	87400	100900	126100
32	29.25	85200	100700	116200	145300
34	31.25	97300	115000	132600	165800
36	33.25	110100	130100	150200	187700
38	35.25	123800	146300	168800	211000
40	37.25	138200	163300	188500	236000
42	39.25	153500	181400	209000	262000
44	41.25	169500	200000	231000	289000
46	43.25	186300	220000	254000	318000
48	45.25	204000	241000	278000	348000
50	47.00	220000	260000	300000	375000
52	49.00	239000	283000	326000	408000
54	51.00	259000	306000	353000	442000
56	53.00	280000	331000	382000	477000
58	55.00	301000	356000	411000	514000
60	57.00	324000	382000	441000	552000

Table 3-26

RESISTANCE FUNCTIONS FOR TWO-WAY ISOTROPIC REINFORCED CONCRETE SLABS, FIXED EDGE SUPPORTS (α = 0.6)

f_{dy} (psi)		44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$		0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.									
ϕ_v = 0.0		11810	21000	16500	29330	21960	39040	34320	61010
ϕ_v = 0.5		5690	10120	7140	12690	8580	15250	11200	19920
ϕ_v = 1.0		3341	5940	3960	7050	4540	8070	5490	9760
ϕ_v = 1.5		2190	3900	2520	4470	2800	4980	3250	5770
Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)							
D(in.)	d(in.)								
10	7.75	5210		6160		7110		8880	
12	9.50	7830		9260		10680		13350	
14	11.50	11480		13560		15650		19560	
16	13.50	15820		18690		21600		27000	
18	15.50	20800		24600		28400		35500	
20	17.50	26600		31400		36200		45300	
22	19.50	33000		39000		45000		56200	
24	21.50	40100		47400		54700		68400	
26	23.50	47900		56600		65300		81700	
28	25.50	56400		66700		76900		96200	
30	27.25	64400		76200		87900		109800	
32	29.25	74200		87700		101200		126600	
34	31.25	84700		100200		115600		144400	
36	33.25	95900		113400		130800		163500	
38	35.25	107800		127400		147000		183800	
40	37.25	120400		142300		164200		205000	
42	39.25	133700		158000		182300		228000	
44	41.25	147700		174500		201000		252000	
46	43.25	162300		191800		221000		277000	
48	45.25	177700		210000		242000		303000	
50	47.00	191700		227000		261000		327000	
52	49.00	208000		246000		284000		355000	
54	51.00	226000		267000		308000		385000	
56	53.00	244000		288000		332000		415000	
58	55.00	263000		310000		358000		447000	
60	57.00	282000		333000		384000		481000	

Table 3-27

RESISTANCE FUNCTIONS FOR TWO-WAY ISOTROPIC REINFORCED CONCRETE SLABS, FIXED EDGE SUPPORTS (α = 0.5)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.								
ϕ_v = 0.0	11060	19660	15450	27470	20570	36570	32140	57140
ϕ_v = 0.5	5330	9480	6690	11890	8030	14280	10500	18660
ϕ_v = 1.0	3130	5560	3710	6600	4250	7560	5140	9140
ϕ_v = 1.5	2050	3650	2360	4190	2620	4660	3040	5410

Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)			
D(in.)	d(in.)				
10	7.75	4660	5510	6360	7950
12	9.50	7000	8280	9550	11940
14	11.50	10260	12130	14000	17500
16	13.50	14140	16720	19290	24100
18	15.50	18650	22000	25400	31800
20	17.50	23800	28100	32400	40500
22	19.50	29500	34900	40200	50300
24	21.50	35900	42400	48900	61200
26	23.50	42900	50700	58400	73100
28	25.50	50500	59600	68800	86000
30	27.25	57600	68100	78600	98200
32	29.25	66400	78500	90500	113200
34	31.25	75800	89600	103400	129200
36	33.25	85800	101400	117000	146300
38	35.25	96400	114000	131500	164400
40	37.25	107700	127300	146900	183600
42	39.25	119600	141300	163000	204000
44	41.25	132100	156100	180100	225000
46	43.25	145200	171600	198000	247000
48	45.25	158900	187800	217000	271000
50	47.00	171400	203000	234000	292000
52	49.00	186300	220000	254000	318000
54	51.00	202000	239000	275000	344000
56	53.00	218000	258000	297000	372000
58	55.00	235000	277000	320000	400000
60	57.00	252000	298000	344000	430000

$\alpha = \frac{L_S}{L_L}$ is transformed to $\alpha' = \frac{L'_S}{L_L} = \frac{\alpha}{\sqrt{\mu}}$ through the relationship,

$$L'_S = \frac{L_S}{\sqrt{\mu}}.$$

The term μ represents the coefficient of orthotropy which, for the two-way orthotropic slabs analyzed in this study, can be considered as the ratio of the area of tension steel in the short-span direction to that in the long-span direction. Thus, for an under-reinforced orthotropic slab of uniform thickness, ultimate moment-capacity in the L_S direction is μ times the ultimate moment capacity in the L_L direction.

For maximum weight economy, it has been recommended[39] that

$$\mu_e = \frac{3 - 2\alpha^2}{\alpha^2}.$$

Substituting this recommended value for μ_e in the equation $\alpha' = \frac{\alpha}{\mu}$, the α transformation associated with maximum weight economy can be obtained.

$$\alpha' = \frac{\alpha^2}{\sqrt{3 - 2\alpha^2}} \qquad (3.34.12)$$

With this transformation, the orthotropic slab may then be analyzed as an equivalent isotropic slab.

(a) Simply-Supported Two-Way Reinforced Orthotropic Slab

The ultimate flexural resistance of a one-inch width of two-way reinforced slab is expressed by Equation 3.34.1 as,

$$q_f = 0.000750(\phi_{Sc} + \phi_{Se}) f_{dy} \left(\frac{d}{L_S}\right)^2 \left[\alpha\left(\frac{\phi_{Lc} + \phi_{Le}}{\phi_{Sc} + \phi_{Se}}\right) + \left(\frac{2 - \alpha}{3 - 2\alpha}\right)\right] \qquad (3.34.1)$$

For an orthotropic slab, $\phi_{Sc} = \mu \phi_{Lc}$ and $\phi_{Se} = \mu \phi_{Le}$. Substituting in Equation 3.34.1 yields,

$$q_f = 0.00075\,(\phi_{Sc} + \phi_{Se}) f_{dy} \left(\frac{d}{L_S}\right)^2 \left[\frac{\alpha}{\mu} + \left(\frac{2-\alpha}{3-2\alpha}\right)\right] \qquad (3.34.13)$$

For the simply-supported slab, with $\phi_{Se} = 0$, Equation 3.34.13 reduces to

$$q_f = 0.00075\,\phi_{Sc} f_{dy} \left(\frac{d}{L_S}\right)^2 \left[\frac{\alpha}{\mu} + \left(\frac{2-\alpha}{3-2\alpha}\right)\right] \qquad (3.34.14)$$

An analytical inconsistency becomes apparent when Equations 3.34.14 and 3.33.3 are compared. While identical values of q_f should be obtained as μ becomes very large, such is not found in actuality. The inconsistency is attributable, it is believed, to a fundamental lack of rationality in the one-way and two-way slab equations. This short-coming becomes emphasized when orthotropic behavior is examined for the two-way slab. Good agreement between the flexural equations for the one-way and two-way slabs can be obtained empirically, if so desired, if the right-hand side of Equation 3.34.14 is multiplied by $1/12\,(5 + 3/\alpha + 4/\mu)$.

By substituting μ_e for μ in Equation 3.34.14, where

$$\mu_e = \left(\frac{3-2\alpha}{\alpha^2}\right)^2 ,$$

we obtain

$$q_f = 0.00075\,\phi_{Sc} f_{dy} \left(\frac{d}{L_S}\right)^2 \left[\frac{\alpha^3}{3-2\alpha^2} + \frac{2-\alpha}{3-2\alpha}\right] \qquad (3.34.15)$$

(for $\mu = \mu_e$)

Following the recommendations of Reference 2, the ultimate resistances of a two-way reinforced isotropic slab to "pure" shear and to diagonal tension-shear compression are taken as $2/3\,(1 + \alpha)$ times those of a comparable one-way reinforced slab. Since the shearing resistance of the slab is not affected by two-way reinforcement, the anticipated increase in ultimate resistance must reflect the load carried by end-walls due to two-way

slab action. The load distribution on the end walls of an isotropic two-way slab is roughly proportional to α. The same ratio for an orthotropic slab will, depending on the particular pattern of orthotropy which is selected, lie somewhere between α and α/μ. Recognizing that a considerable degree of approximation is involved, the equations for q_v and q_{sc} for the two-way isotropic slab[2] will be modified, through introducing the factor μ, before they are applied to the two-way orthotropic slab.

The ultimate resistance of a one-inch width of two-way reinforced orthotropic slab to diagonal tension and shear compression, assuming simply-supported edges and no web reinforcement, can then be expressed as,

$$q_{sc} = \frac{0.482}{(2 + \theta')} \left(\frac{d}{L_S}\right)^2 (f'_c \phi_{Sc})^{1/2} \left(1.5 + \frac{\alpha}{2\mu}\right) \qquad (3.34.16a)$$

Equation 3.34.16a is comparable to Equation 3.34.4b, which describes the q_{sc} mode for the simply-supported isotropic slab. For the orthotropic slab, however, the term $(1.5 + \frac{\alpha}{2\mu})$ is substituted for the $(1 + \alpha)$ term which appears in the isotropic equation. This approach assumes that the orthotropic slab has the shearing-mode resistance of a one-way slab, plus an effective additional resistance proportional to $(\frac{\alpha}{2\mu})$ due to its two-way action. Since the values of μ associated with maximum weight economy increase rapidly as α is decreased,[39] the orthotropic shearing-mode resistance expressed by Equation 3.34.16a becomes essentially that of a one-way slab as α approaches a value of 0.5. At the other extreme, a value of $\alpha = 1.0$ corresponds to the isotropic case with $\mu_e = 1.0$. Equation 3.34.4b should thus be used in lieu of Equation 3.34.16a when $\alpha = 1.0$.

If web reinforcement is supplied in the form of vertical stirrups, Equation 3.34.16a becomes, for $0.5 \leq \alpha \leq 0.9$

$$q_{sc} = \left[\frac{0.482 + 0.00000964\phi_v f_{dy}}{(2 + \theta')}\right] (f'_c \phi_{Sc})^{1/2} \left(\frac{d}{L_S}\right)^2 \left(1.5 + \frac{\alpha}{2\mu}\right) \qquad (3.34.16b)$$

By introducing $\mu = \mu_e$, Equation 3.34.16a and 3.34.16b become, for $0.5 \leq \alpha \leq 0.9$,

$$q_{sc} = \frac{0.482}{(2 + \theta')} \left(\frac{d}{L_S}\right)^2 (f'_c \phi_{Sc})^{1/2} \left(1.5 + \frac{\alpha^3}{6 - 4\alpha^2}\right) \qquad (3.34.17a)$$

(for $\mu = \mu_e$)

$$q_{sc} = \left[\frac{0.482 + 0.00000964\ \phi_v f_{dy}}{(2 + \theta')}\right] (f'_c \phi_{Sc})^{1/2} \left(\frac{d}{L_S}\right)^2 \left(1.5 + \frac{\alpha^3}{6-4\alpha^2}\right) \quad (3.34.17b)$$

(for $\mu = \mu_e$)

The ultimate shearing resistance of a one-inch width of two-way reinforced orthotropic slab with simply-supported edges, following the same reasoning used in developing Equation 3.34.16, can be expressed approximately as follows for $0.5 \leqslant \alpha \leqslant 0.9$.

$$q_v = 0.0245 \left(1.5 + \frac{\alpha}{2\mu}\right) f'_c \left(\frac{d}{L_S}\right) \quad (3.34.18)$$

By substituting $\mu = \mu_e$, Equation 3.34.18 becomes

$$q_v (\text{for } \mu = \mu_e) = 0.0245 \left(1.5 + \frac{\alpha^3}{6 - 4\alpha^2}\right) f'_c \left(\frac{d}{L_S}\right) \quad (3.34.19)$$

Equations 3.34.14, 3.34.16 and 3.34.18 express the ultimate resistances of a one-inch width of two-way reinforced orthotropic slab, simply supported, to the three failure modes considered in this study. Equations 3.34.15, 3.34.17 and 3.34.19 provide similar information and include the assumption that the ratio of steel in the short-span direction to that in the long-span direction, $\left(\phi_S / \phi_L = \mu\right)$, will satisfy the relation $\mu = \mu_e$, where

$$\mu_e = \left(\frac{3-2\alpha^2}{\alpha^2}\right)$$

The expressions for q_f and q_{sc} can be solved simultaneously to obtain values of ϕ_{Sc} associated with equal ultimate resistances in flexure and in diagonal tension or shear compression.

From Equation 3.34.14 and 3.34.16,

$$\phi_{Sc} = \left[\frac{1450 + 0.0289\ \phi_v f_{dy}}{f_{dy}\ (2 + \theta')}\right]^2 \left[\frac{3\mu + \alpha}{4.5\alpha + 4.5\mu\left(\frac{2-\alpha}{3-2\alpha}\right)}\right]^2 f'_c \quad (3.34.20)$$

From Equations 3.34.15 and 3.34.17b,

$$\phi_{Sc} = \left[\frac{1450 + 0.0289\,\phi_v f_{dy}}{f_{dy}(2+\theta')}\right]^2 f'_c \left[\frac{9/\alpha^2 - 6 + \alpha}{4.5\alpha + 4.5\left(\frac{3-2\alpha^2}{\alpha^2}\right)\left(\frac{2-\alpha}{3-2\alpha}\right)}\right]^2 \qquad (3.34.21)$$

(for $\mu = \mu_e$)

This expression, which yields values of ϕ_{Sc} corresponding to equal ultimate resistances for the two-way orthotropic slab in flexure and in diagonal tension or shear compression, can be directly related to the comparable expressions for ϕ_c in the one-way slab. Assuming $\mu = \mu_e$, Equation 3.33.8b and 3.34.21 yield,

$$\phi_{Sc} = \phi_c \left[\frac{9/\alpha^2 - 6 + \alpha}{4.5\left\{\alpha + \left(\frac{3-2\alpha^2}{\alpha^2}\right)\left(\frac{2-\alpha}{3-2\alpha}\right)\right\}}\right]^2 \qquad (3.34.22)$$

Values of ϕ_{Sc} obtained form Equations 3.34.20 or 3.34.21 must be checked, by use of Equations 3.34.18 or 3.34.19, to ensure that slab resistance to "pure" shear will not control the design. The maximum permissible value of q'_d, if "pure" shear is not to govern, can be expressed for the two-way orthotropic simply-supported slab, as

$$q'_d\,(\text{max.}) = \frac{0.1635}{(d/L_S)} \left[\frac{3\mu + \alpha}{\alpha + \mu\left(\frac{2-\alpha}{3-2\alpha}\right)}\right] \qquad (3.34.23a)$$

$$q'_d\,(\text{max.} @ \mu = \mu_e) = \frac{0.1635}{(d/L_S)} \left[\frac{9/\alpha^2 - 6 + \alpha}{\alpha + \left(\frac{3-2\alpha^2}{\alpha^2}\right)\left(\frac{2-\alpha}{3-2\alpha}\right)}\right] \qquad (3.34.23b)$$

As in earlier sections, $q'_d = pf_{dy}/f'_c$. Representative values from Equation 3.34.23b are plotted on Figure 3-4.

Values used for ϕ_{Sc} must also lie within acceptable maximum and minimum limits, as previously explained. Tables 3-28 to 3-32 contain resistance functions calculated for $\alpha = 0.9$, 0.8, 0.7, 0.6 and 0.5. Orthotropic two-way reinforced slabs with simply-supported edges can be designed with the aid of these resistance functions, as described for one-way reinforced slabs, for cases where "pure" shear does not control. Tables are not included for $\alpha = 1.0$, since the orthotropic and isotropic slabs with $\mu = \mu_e$ are identical for this limiting case. It should be noted that the resistance function of Tables 3-28 to 3-32 are computed by assuming

$$\mu = \mu_e = \left(\frac{3-2\alpha^2}{\alpha^2} \right).$$

(b) <u>Fixed-Edge Two-Way Reinforced Orthotropic Slabs</u>

By assuming that equal areas of reinforcement will be provided in the top and bottom of the slab, $\phi_{Sc} = \phi_{Se}$ and $\phi_{Lc} = \phi_{Le}$, then Equation 3.34.13, as applied to a one-inch width of fixed-edge slab, becomes

$$q_f = 0.0015\, \phi_{Sc} f_{dy} \left(\frac{d}{L_S} \right)^2 \left[\frac{\alpha}{\mu} + \frac{2-\alpha}{3-2\alpha} \right] \qquad (3.34.24)$$

As explained for the simply-supported orthotropic slab, there is a basic inconsistency between Equations 3.34.24 and 3.33.10 for large values of μ. The numerical discrepancy in predicted values of q_f can be empirically reduced if the right hand side of Equation 3.34.24 is multiplied by 1/12 $\left[5 + 3/\alpha + 4/\mu\right]$.

By substituting $\mu = \mu_e$ in Equation 3.34.22 where $\mu_e = \dfrac{3-2\alpha^2}{\alpha^2}$, we obtain

$$q_f = 0.0015\, \phi_{Sc} f_{dy} \left(\frac{d}{L_S} \right)^2 \left[\frac{\alpha^3}{3-2\alpha^2} + \frac{2-\alpha}{3-2\alpha} \right] \qquad (3.34.25$$

(for $\mu = \mu_e$)

The resistance of a one-inch width of two-way reinforced orthotropic slab to diagonal tension and shear compression, assuming fixed edges and no web reinforcement, can be expressed for $0.5 \lesssim \alpha \lesssim 0.9$ as

Table 3-28

RESISTANCE FUNCTIONS FOR TWO-WAY ORTHOTROPIC REINFORCED CONCRETE SLABS, SIMPLY SUPPORTED (α = 0.9)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\Theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.								
ϕ_v = 0.0	21110	37520	29480	52410	39250	69770	61320	109020
ϕ_v = 0.5	10200	18130	12790	22740	15370	27320	20080	35700
ϕ_v = 1.0	5990	10650	7110	12640	8140	14470	9850	17510
ϕ_v = 1.5	3930	7000	4520	8030	5030	8940	5830	10370

Required Depth of Beam		Resistance Function $\frac{q_f L^2_S}{\phi_{Sc}} = \frac{q_{sc} L^2_S}{\phi_{Sc}}$ (psi-sq ft)			
D(in.)	d(in.)	44,000	52,000	60,000	75,000
10	7.75	3050	3600	4150	5190
12	9.50	4580	5410	6240	7800
14	11.50	6710	7930	9150	11430
16	13.50	9240	10920	12610	15760
18	15.50	12190	14400	16620	20800
20	17.50	15530	18360	21200	26500
22	19.50	19290	22800	26300	32900
24	21.50	23400	27700	32000	40000
26	23.50	28000	33100	38200	47700
28	25.50	33000	39000	45000	56200
30	27.25	37700	44500	51400	64200
32	29.25	43400	51300	59200	74000
34	31.25	49500	58500	67500	84400
36	33.25	56100	66300	76500	95600
38	35.25	63000	74500	85900	107400
40	37.25	70400	83200	96000	120000
42	39.25	78100	92300	106600	133200
44	41.25	86300	102000	117700	147100
46	43.25	94900	112100	129400	161700
48	45.25	103900	122700	141600	177000
50	47.00	112000	132400	152800	191000
52	49.00	121800	143900	166100	208000
54	51.00	131900	155900	179900	225000
56	53.00	142500	168400	194300	243000
58	55.00	153400	181300	209000	262000
60	57.00	164800	194800	225000	281000

Table 3-29

RESISTANCE FUNCTIONS FOR TWO-WAY ORTHOTROPIC REINFORCED CONCRETE SLABS, SIMPLY SUPPORTED (α = 0.8)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.								
ϕ_v = 0.0	18690	33220	26100	46410	34750	61780	54300	96540
ϕ_v = 0.5	9030	16060	11330	20130	13600	24200	17780	31610
ϕ_v = 1.0	5300	9430	6290	11190	7210	12810	8720	15510
ϕ_v = 1.5	3480	6200	4000	7110	4450	7910	5160	9180

Required Depth of Beam D(in.)	d(in.)	Resistance Function $\frac{q_f L^2 S}{\phi_{Sc}} = \frac{q_{sc} L^2 S}{\phi_{Sc}}$ (psi-sq ft)			
10	7.75	2510	2970	3430	4290
12	9.50	3780	4460	5150	6440
14	11.50	5540	6540	7550	9440
16	13.50	7630	9020	10400	13000
18	15.50	10060	11890	13710	17140
20	17.50	12820	15150	17480	21900
22	19.50	15920	18810	21700	27100
24	21.50	19350	22900	26400	33000
26	23.50	23100	27300	31500	39400
28	25.50	27200	32200	37100	46400
30	27.25	31100	36700	42400	53000
32	29.25	35800	42300	48800	61000
34	31.25	40900	48300	55700	69700
36	33.25	46300	54700	63100	78900
38	35.25	52000	61500	70900	88700
40	37.25	58100	68600	79200	99000
42	39.25	64500	76200	87900	109900
44	41.25	71200	84200	97100	121400
46	43.25	78300	92500	106800	133500
48	45.25	85700	101300	116900	146100
50	47.00	92500	109300	126100	157600
52	49.00	100500	118800	137100	171300
54	51.00	108900	128700	148500	185600
56	53.00	117600	139000	160300	200000
58	55.00	126600	149700	172700	216000
60	57.00	136000	160700	185500	232000

Table 3-30

RESISTANCE FUNCTIONS FOR TWO-WAY ORTHOTROPIC REINFORCED CONCRETE SLABS, SIMPLY SUPPORTED (α = 0.7)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\Theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.								
ϕ_v = 0.0	16640	29580	23240	41320	30940	55010	48350	85950
ϕ_v = 0.5	8040	14300	10080	17930	12120	21540	15830	28150
ϕ_v = 1.0	4720	8400	5600	9960	6420	11410	7770	13810
ϕ_v = 1.5	3100	5520	3560	6330	3960	7050	4600	8170

Required Depth of Beam D(in.)	Required Depth of Beam d(in.)	Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)			
10	7.75	1980	2340	2700	3380
12	9.50	2980	3520	4060	5080
14	11.50	4360	5160	5950	7440
16	13.50	6010	7110	8200	10250
18	15.50	7930	9370	10810	13510
20	17.50	10110	11940	13780	17230
22	19.50	12550	14830	17110	21400
24	21.50	15250	18030	20800	26000
26	23.50	18220	21500	24900	31100
28	25.50	21500	25400	29300	36600
30	27.25	24500	29000	33400	41800
32	29.25	28200	33400	38500	48100
34	31.25	32200	38100	43900	54900
36	33.25	36500	43100	49800	62200
38	35.25	41000	48500	55900	69900
40	37.25	45800	54100	62400	78100
42	39.25	50800	60100	69300	86700
44	41.25	56200	66400	76600	95700
46	43.25	61700	73000	84200	105200
48	45.25	67600	79900	92100	115200
50	47.00	72900	86200	99400	124300
52	49.00	79200	93600	108000	135100
54	51.00	85800	101400	117000	146300
56	53.00	92700	109600	126400	158000
58	55.00	99800	118000	136100	170200
60	57.00	107200	126700	146200	182800

Table 3-31

RESISTANCE FUNCTIONS FOR TWO-WAY ORTHOTROPIC REINFORCED CONCRETE SLABS, SIMPLY SUPPORTED (α = 0.6)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
	Read value of k. Compute $\phi_{Sc} = f'_c/k$.							
ϕ_v = 0.0	14990	26650	20940	37220	27870	49560	43550	77430
ϕ_v = 0.5	7240	12880	9080	16150	10920	19410	14260	25360
ϕ_v = 1.0	4250	7560	5050	8980	5780	10280	7000	12440
ϕ_v = 1.5	2790	4970	3210	5700	3570	6350	4140	7360

Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)			
D(in.)	d(in.)	44,000	52,000	60,000	75,000
10	7.75	1840	2170	2500	3130
12	9.50	2760	3260	3760	4700
14	11.50	4040	4780	5510	6890
16	13.50	5570	6580	7590	9490
18	15.50	7340	8680	10010	12510
20	17.50	9360	11060	12760	15950
22	19.50	11620	13730	15840	19800
24	21.50	14120	16690	19260	24100
26	23.50	16870	19940	23000	28800
28	25.50	19870	23500	27100	33900
30	27.25	22700	26800	30900	38700
32	29.25	26100	30900	35600	44600
34	31.25	29800	35300	40700	50900
36	33.25	33800	39900	46100	57600
38	35.25	38000	44900	51800	64700
40	37.25	42400	50100	57800	72300
42	39.25	47100	55600	64200	80200
44	41.25	52000	61400	70900	88600
46	43.25	57200	67500	77900	97400
48	45.25	62600	73900	85300	106600
50	47.00	67500	79800	92000	115100
52	49.00	73400	86700	100000	125100
54	51.00	79500	93900	108400	135500
56	53.00	85800	101400	117000	146300
58	55.00	92400	109200	126000	157600
60	57.00	99300	117300	135400	169200

Table 3-32

RESISTANCE FUNCTIONS FOR TWO-WAY ORTHOTROPIC REINFORCED CONCRETE SLABS, SIMPLY SUPPORTED (α = 0.5)

f_{dy} (psi)		44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$		0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.									
ϕ_v = 0.0		13700	24350	19130	34020	25470	45280	39800	70760
ϕ_v = 0.5		6620	11770	8300	14760	9980	17740	13040	23170
ϕ_v = 1.0		3890	6910	4610	8200	5280	9390	6390	11370
ϕ_v = 1.5		2550	4540	2930	5210	3260	5800	3780	6730
Required Depth of Beam		Resistance Function $\frac{q_f L^2_S}{\phi_{Sc}} = \frac{q_{sc} L^2_S}{\phi_{Sc}}$ (psi-sq ft)							
D(in.)	d(in.)								
10	7.75	1610		1910		2200		2750	
12	9.50	2430		2870		3310		4140	
14	11.50	3560		4200		4850		6060	
16	13.50	4900		5790		6680		8350	
18	15.50	6460		7630		8810		11010	
20	17.50	8230		9730		11230		14040	
22	19.50	10220		12080		13940		17430	
24	21.50	12430		14690		16950		21200	
26	23.50	14850		17550		20200		25300	
28	25.50	17480		20700		23800		29800	
30	27.25	19970		23600		27200		34000	
32	29.25	23000		27200		31400		39200	
34	31.25	26300		31000		35800		44800	
36	33.25	29700		35100		40500		50700	
38	35.25	33400		39500		45600		57000	
40	37.25	37300		44100		50900		63600	
42	39.25	41400		49000		56500		70600	
44	41.25	45800		54100		62400		78000	
46	43.25	50300		59400		68600		85700	
48	45.25	55100		65100		75100		93800	
50	47.00	59400		70200		81000		101200	
52	49.00	64600		76300		88000		110000	
54	51.00	69900		82700		95400		119200	
56	53.00	75500		89300		103000		128700	
58	55.00	81300		96100		110900		138600	
60	57.00	87400		103200		119100		148900	

$$q_{sc} = \frac{1.175}{(2+\theta')}\left(\frac{d}{L_S}\right)^2 (f'_c \phi_{Sc})^{1/2}\left(1.5+\frac{\alpha}{2\mu}\right) \qquad (3.34.26a)$$

If web reinforcement is supplied in the form of vertical stirrups, Equation 3.34.26a becomes

$$q_{sc} = \left[\frac{1.175+0.0000235\,\phi_v f_{dy}}{2+\theta'}\right](f'_c \phi_{Sc})^{1/2}\left(\frac{d}{L_S}\right)^2\left(1.5+\frac{\alpha}{2\mu}\right) \qquad (3.34.26b)$$

By introducing $\mu = \mu_e$, Equations 3.34.26a and 3.34.26b become

$$q_{sc} = \frac{1.175}{(2+\theta')}\left(\frac{d}{L_S}\right)^2 (f'_c \phi_{Sc})^{1/2}\left(1.5+\frac{\alpha^3}{6-4\alpha^2}\right) \qquad (3.34.27a)$$
(for $\mu = \mu_e$)

$$q_{sc} = \left[\frac{1.175+0.0000235\,\phi_v f_{dy}}{(2+\theta')}\right](f'_c \phi_{Sc})^{1/2}\left(\frac{d}{L_S}\right)^2\left(1.5+\frac{\alpha^3}{6-4\alpha^2}\right) \qquad (3.34.27b)$$
(for $\mu = \mu_c$)

The ultimate shearing resistance of the fixed-edge orthotropic slab is equal to that determined for the simply-supported orthotropic slab. For $0.5 \leq \alpha \leq 0.9$ this resistance can be expressed as follows;

$$q_v = 0.0245\left(1.5+\frac{\alpha}{2\mu}\right) f'_c \left(\frac{d}{L_S}\right) \qquad (3.34.18)$$

$$q_v\ (\text{for } \mu = \mu_e) = 0.0245\left(1.5+\frac{\alpha^3}{6-4\alpha^2}\right) f'_c \left(\frac{d}{L_S}\right) \qquad (3.34.19)$$

Equations 3.34.24, 3.34.26 and 3.34.18 express the ultimate resistances of a one-inch width of two-way reinforced orthotropic slab, assuming fixed-edge support, to the three failure modes considered in this study. Equations 3.34.25, 3.34.29 and 3.34.19 provide similar information and include the assumption that

$$\mu = \mu_e = \left(\frac{3-2\alpha^2}{\alpha^2}\right)$$

Solving Equations 3.34.24 and 3.34.26 we obtain the values of ϕ_{Sc} associated with equal ultimate resistances in flexure and in diagonal tension or shear compression.

$$\phi_{Sc} = \left[\frac{1765+0.0353\,\phi_v f_{dy}}{f_{dy}\,(2+\theta')}\right]^2 \left[\frac{3\mu+\alpha}{4.5\alpha + 4.5\mu\left(\frac{2-\alpha}{3-2\alpha}\right)}\right]^2 f'_c \qquad (3.34.28)$$

Similarly, from Equations 3.34.25 and 3.34.27,

$$\phi_{Sc} = \left[\frac{1765+0.0353\,\phi_v f_{dy}}{f_{dy}(2+\theta')}\right]^2 \left[\frac{9/\alpha^2 - 6 + \alpha}{4.5\alpha + 4.5\left(\frac{3-2\alpha^2}{\alpha^2}\right)\left(\frac{2-\alpha}{3-\alpha}\right)}\right]^2 \qquad (3.34.29)$$

$(\mu = \mu_e)$

Values of ϕ_{Sc} obtained from Equations 3.34.28 or 3.34.29 must be checked by use of Equation 3.34.18 or 3.34.19 to ensure that resistance in "pure" shear will not control the design. The maximum permissible value of q'_d, if "pure" shear is not to govern, can be expressed for the two-way orthotropic fixed-edge slab as

$$q'_d(\text{max.}) = \frac{0.0818}{(d/L_S)}\left[\frac{3\mu + \alpha}{\alpha + \mu\left(\frac{2-\alpha}{3-2\alpha}\right)}\right] \qquad (3.34.30a)$$

$$q'_d(\text{max.} @ \mu = \mu_e) = \frac{0.0818}{(d/L_S)}\left[\frac{9/\alpha^2 - 6 + \alpha}{\alpha + \left(\frac{3-2\alpha^2}{\alpha^2}\right)\left(\frac{2-\alpha}{3-2\alpha}\right)}\right] \qquad (3.34.30b)$$

Representative values form Equation 3.34.30b are plotted on Figure 3-4. Values of ϕ_{Sc} must still lie within acceptable maximum and minimum limits. Tables 3-33 to 3-37 contain resistance functions calculated for α = 0.9, 0.8, 0.7, 0.6 and 0.5 with

$$\mu = \mu_e = \left(\frac{3-2\alpha^2}{\alpha^2}\right)$$

Orthotropic two-way reinforced slabs with fixed-edge support can be designed with the aid of these resistance functions, assuming "pure" shear does not control, by following the procedures described for one-way reinforced slabs.

3.34.4 Cost Studies

Cost studies for one-way reinforced concrete slabs, specifically referenced to a condition of full edge-fixity, are presented in Section 3.33.2 In the following paragraphs, a generalized cost analysis will be developed which, with the substitution of proper coefficients, can be applied to any type of reinforced concrete slab. The analytical expressions which describe ultimate slab resistances in the three postulated failure modes will first be related, for the several types of reinforcement and conditions of end restraint, through the coefficients k_f, k_v, and k_{sc} where

k_f = flexure resistance coefficient

k_v = shear resistance coefficient

k_{sc} = diagonal tension resistance coefficient

Table 3-33

RESISTANCE FUNCTIONS FOR TWO-WAY ORTHOTROPIC REINFORCED CONCRETE SLABS, FIXED EDGE SUPPORTS (α = 0.9)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.								
ϕ_v = 0.0	14250	25330	19900	35380	26490	47090	41390	73590
ϕ_v = 0.5	6880	12240	8630	15350	10370	18440	13550	24100
ϕ_v = 1.0	4040	7190	4800	8530	5490	9770	6650	11820
ϕ_v = 1.5	2650	4720	3050	5420	3400	6030	3940	7000

Required Depth of Beam		Resistance Function $\dfrac{q_f L_S^2}{\phi_{Sc}} = \dfrac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)			
D(in.)	d(in.)				
10	7.75	6090	7200	8310	10390
12	9.50	9160	10820	12480	15610
14	11.50	13420	15860	18290	22900
16	13.50	18490	21800	25200	31500
18	15.50	24400	28800	33200	41500
20	17.50	31100	36700	42400	53000
22	19.50	38600	45600	52600	65800
24	21.50	46900	55400	63900	79900
26	23.50	56000	66200	76400	95500
28	25.50	66000	78000	90000	112400
30	27.25	75300	89000	102700	128400
32	29.25	86800	102600	118400	147900
34	31.25	99100	117100	135100	168900
36	33.25	112200	132500	152900	191200
38	35.25	126100	149000	171900	215000
40	37.25	140800	166400	191900	240000
42	39.25	156300	184700	213000	266000
44	41.25	172600	204000	235000	294000
46	43.25	189800	224000	259000	323000
48	45.25	208000	245000	283000	354000
50	47.00	224000	265000	306000	382000
52	49.00	244000	288000	332000	415000
54	51.00	264000	312000	360000	450000
56	53.00	285000	337000	389000	486000
58	55.00	307000	363000	418000	523000
60	57.00	330000	390000	449000	562000

Table 3-34

RESISTANCE FUNCTIONS FOR TWO-WAY ORTHOTROPIC REINFORCED CONCRETE SLABS, FIXED EDGE SUPPORTS (α = 0.8)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\Theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.								
ϕ_v = 0.0	12620	22420	17620	31330	23460	41700	36650	65160
ϕ_v = 0.5	6100	10840	7650	13590	9180	16340	12000	21340
ϕ_v = 1.0	3580	6360	4240	7550	4870	8650	5890	10470
ϕ_v = 1.5	2350	4180	2700	4800	3000	5340	3480	6200

Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)			
D(in.)	d(in.)				
10	7.75	5030	5940	6860	8570
12	9.50	7560	8930	10300	12880
14	11.50	11070	13090	15100	18870
16	13.50	15260	18030	20800	26000
18	15.50	20100	23800	27400	34300
20	17.50	25600	30300	35000	43700
22	19.50	31800	37600	43400	54300
24	21.50	38700	45700	52800	66000
26	23.50	46200	54600	63000	78800
28	25.50	54400	64300	74200	92800
30	27.25	62200	73500	84800	106000
32	29.25	71600	84700	97700	122100
34	31.25	81800	96600	111500	139400
36	33.25	92600	109400	126200	157800
38	35.25	104000	122900	141900	177300
40	37.25	116200	137300	158400	198000
42	39.25	129000	152400	175900	220000
44	41.25	142500	168400	194300	243000
46	43.25	156600	185100	214000	267000
48	45.25	171400	203000	234000	292000
50	47.00	184900	219000	252000	315000
52	49.00	201000	238000	274000	343000
54	51.00	218000	257000	297000	371000
56	53.00	235000	278000	321000	401000
58	55.00	253000	299000	345000	432000
60	57.00	272000	321000	371000	464000

Table 3-35

RESISTANCE FUNCTIONS FOR TWO-WAY ORTHOTROPIC REINFORCED CONCRETE SLABS, FIXED EDGE SUPPORTS (α = 0.7)

f_{dy} (psi)		44,000		52,000		60,000		75,000	
$\Theta' = \phi'/\phi_{Sc}$		0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.									
ϕ_v = 0.0		11230	19970	15690	27890	20880	37130	32640	58020
ϕ_v = 0.5		5430	9650	6800	12100	8180	14540	10680	19000
ϕ_v = 1.0		3190	5670	3780	6720	4330	7700	5240	9320
ϕ_v = 1.5		2090	3730	2400	4270	2670	4760	3100	5510
Required Depth of Beam		Resistance Function $\frac{q_f L^2_S}{\phi_{Sc}} = \frac{q_{sc} L^2_S}{\phi_{Sc}}$ (psi-sq ft)							
D(in.)	d(in.)								
10	7.75	3960		4680		5410		6760	
12	9.50	5960		7040		8120		10150	
14	11.50	8730		10320		11900		14880	
16	13.50	12030		14220		16400		20500	
18	15.50	15860		18740		21600		27000	
20	17.50	20200		23900		27600		34500	
22	19.50	25100		29700		34200		42800.	
24	21.50	30500		36100		41600		52000	
26	23.50	36400		43100		49700		62100	
28	25.50	42900		50700		58500		73200	
30	27.25	49000		57900		66800		83500	
32	29.25	56500		66700		77000		96300	
34	31.25	64500		76200		87900		109900	
36	33.25	73000		86200		99500		124400	
38	35.25	82000		96900		111800		139800	
40	37.25	91600		108200		124900		156100	
42	39.25	101700		120200		138700		173300	
44	41.25	112300		132700		153100		191400	
46	43.25	123500		145900		168400		210000	
48	45.25	135100		159700		184300		230000	
50	47.00	145800		172300		198800		249000	
52	49.00	158500		187300		216000		270000	
54	51.00	171700		203000		234000		293000	
56	53.00	185400		219000		253000		316000	
58	55.00	199600		236000		272000		340000	
60	57.00	214000		253000		292000		366000	

Table 3-36

RESISTANCE FUNCTIONS FOR TWO-WAY ORTHOTROPIC REINFORCED CONCRETE SLABS, FIXED EDGE SUPPORTS (α = 0.6)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.								
ϕ_v = 0.0	10120	17990	14130	25120	18810	33450	29400	52260
ϕ_v = 0.5	4890	8690	6130	10900	7370	13100	9620	17120
ϕ_v = 1.0	2870	5100	3410	6060	3900	6940	4720	8400
ϕ_v = 1.5	1880	3350	2170	3850	2410	4290	2790	4970

Required Depth of Beam D(in.)	Required Depth of Beam d(in.)	Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)			
10	7.75	3670	4340	5010	6260
12	9.50	5520	6520	7520	9400
14	11.50	8080	9550	11020	13780
16	13.50	11140	13160	15190	18980
18	15.50	14680	17350	20000	25000
20	17.50	18720	22100	25500	31900
22	19.50	23200	27500	31700	39600
24	21.50	28200	33400	38500	48200
26	23.50	33700	39900	46000	57500
28	25.50	39700	47000	54200	67700
30	27.25	45400	53600	61900	77400
32	29.25	52300	61800	71300	89100
34	31.25	59700	70500	81400	101700
36	33.25	67600	79800	92100	115200
38	35.25	75900	89700	103500	129400
40	37.25	84800	100200	115600	144500
42	39.25	94100	111300	128400	160500
44	41.25	104000	122900	141800	177200
46	43.25	114300	135100	155900	194900
48	45.25	125100	147900	170600	213000
50	47.00	135000	159500	184100	230000
52	49.00	146700	173400	200000	250000
54	51.00	158900	187800	217000	271000
56	53.00	171700	203000	234000	293000
58	55.00	184900	218000	252000	315000
60	57.00	198500	235000	271000	338000

Table 3-37

RESISTANCE FUNCTIONS FOR TWO-WAY ORTHOTROPIC REINFORCED CONCRETE SLABS, FIXED EDGE SUPPORTS (α = 0.5)

f_{dy} (psi)	44,000		52,000		60,000		75,000	
$\theta' = \phi'/\phi_{Sc}$	0.25	1.00	0.25	1.00	0.25	1.00	0.25	1.00
Read value of k. Compute $\phi_{Sc} = f'_c/k$.								
ϕ_v = 0.0	9250	16440	12910	22960	17190	30560	26860	47760
ϕ_v = 0.5	4470	7940	5600	9960	6740	11970	8800	15640
ϕ_v = 1.0	2620	4660	3110	5540	3560	6340	4310	7670
ϕ_v = 1.5	1720	3060	1980	3520	2200	3920	2550	4540

Required Depth of Beam		Resistance Function $\frac{q_f L_S^2}{\phi_{Sc}} = \frac{q_{sc} L_S^2}{\phi_{Sc}}$ (psi-sq ft)			
D(in.)	d(in.)	44,000	52,000	60,000	75,000
10	7.75	3230	3820	4400	5510
12	9.50	4850	5740	6620	8270
14	11.50	7110	8410	9700	12120
16	13.50	9800	11580	13360	16710
18	15.50	12920	15270	17620	22000
20	17.50	16470	19460	22500	28100
22	19.50	20400	24200	27900	34900
24	21.50	24900	29400	33900	42400
26	23.50	29700	35100	40500	50600
28	25.50	35000	41300	47700	59600
30	27.25	39900	47200	54500	68100
32	29.25	46000	54400	62700	78400
34	31.25	52500	62100	71600	89500
36	33.25	59500	70300	81100	101300
38	35.25	66800	79000	91100	113900
40	37.25	74600	88200	101800	127200
42	39.25	82800	97900	113000	141200
44	41.25	91500	108100	124800	156000
46	43.25	100600	118900	137200	171500
48	45.25	110100	130100	150200	187700
50	47.00	118800	140400	162000	202000
52	49.00	129100	152600	176100	220000
54	51.00	139900	165300	190700	238000
56	53.00	151100	178500	206000	257000
58	55.00	162700	192300	222000	277000
60	57.00	174700	206000	238000	298000

The ultimate flexural resistance of a one-way reinforced, simply-supported slab is obtained from Equation 3.33.3.

$$q_f = 0.00050\,\phi_c\, f_{dy} \left(\frac{d}{L}\right)^2 \qquad (3.34.31)$$

or

$$q_f = k_f\,\phi_c\, f_{dy} \left(\frac{d}{L}\right)^2 \qquad (3.34.32)$$

where

k_f = 0.00050 for a one-way reinforced, simply-supported slab

From Equation 3.34.1 it is apparent that the ultimate flexural resistance for any slab can be expressed as a multiple of the flexural resistance for the simply-supported one-way slab. While it would be desirable to express this relationship through a continuous equation, the empirical constants introduced by Equations 3.31.1 and 3.34.1 preclude a completely rational expression. However, Equations 3.31.2, 3.33.10, 3.34.3, 3.34.9, 3.34.15 and 3.34.25 can readily be solved to obtain numerical values for k_f. These values can be substituted in Equation 3.34.32, which can then be considered as a general expression for the ultimate flexural resistance of any reinforced concrete slab. For two-way slabs, however, ϕ_c and L in Equation 3.34.32 are replaced by ϕ_{Sc} and L_S.

The expression for the ultimate resistance of a simply-supported one-way reinforced slab to diagonal tension or shear compression stresses can be obtained from Equation 3.33.5a.

$$q_{sc} = \frac{0.725}{(2+\theta)} \left(\frac{d}{L}\right)^2 (f'_c\,\phi_c)^{1/2} \left[1 + 0.00002\,\phi_v\, f_{dy}\right] \qquad (3.34.33)$$

By introducing the constant k_{sc}, Equation 3.34.33 can be expressed as a general relationship for any slab.

$$q_{sc} = \frac{k_{sc}}{(2+\theta')}\left(\frac{d}{L}\right)^2 (f'_c \phi_c)^{1/2} \left[1 + 0.00002\,\phi_v f_{dy}\right] \qquad (3.34.34)$$

where

k_{sc} = 0.725 for the simply-supported, one-way reinforced slab

From Equations 3.33.12a, 3.34.5a, 3.34.10b, 3.34.17b and 3.34.27b consistent values of k_{sc} can be obtained for other types of reinforcement and conditions of edge restraint. Thus, in the general case, Equation 3.34.34 can be applied to any slab by introducing appropriate values of k_{sc}. Again, for the two-way slab, $\phi_c = \phi_{Sc}$ and $L = L_S$.

The ultimate resistance of a one-way reinforced, simply-supported slab in "pure" shear is obtained from Equation 3.33.7b.

$$q_v = 0.0367 f'_c \left(\frac{d}{L}\right) \qquad (3.34.35)$$

As before, a general constant k_v can be introduced into this equation.

$$q_v = k_v f'_c \left(\frac{d}{L}\right) \qquad (3.34.36)$$

where

k_v = 0.0367 for a one-way reinforced, simply-supported slab

From Equations 3.33.7b, 3.34.6 and 3.34.19, values of k_v can be obtained for other types of reinforcement and conditions of edge restraint.

The form of the general expression for q_v is thus identical with Equation 3.34.36, with the substitution of appropriate values for k_v. Again, ϕ_c and L are replaced by ϕ_{Sc} and L_S for two-way reinforced slabs. For convenient reference, values of k_f, k_{sc} and k_v for one-way and two-way slabs are listed in Table 3-38, together with values of μ_e for two-way orthotropic slabs.

A generalized cost equation, whose individual terms will include these coefficients, can now be written for a unit area of reinforced concrete slab. The resulting equation, which relates the total in-place cost per square foot of slab to the costs of the individual material components, is identical in form to the generalized cost equation for the one-way reinforced, fixed-edge slab.

$$C_t = C_c + C_s + C_v + C_{st} + C_f \qquad (3.33.29)$$

However, the generalized expressions for the individual cost items differ somewhat from those derived for the one-way, fixed-edge slab.

(1) Concrete

The cost of concrete per square foot of slab is unchanged from the cost supplied in Section 3.33.3 for the one-way reinforced slab.

$$C_c = \frac{X_c D}{12} \qquad (3.33.30a)$$

From Equation 3.34.32, it is apparent that the depth of a reinforced slab can be related to ϕ_{Sc}, assuming that the slab is loaded to its ultimate capacity in flexure.

$$d = \sqrt{\frac{q_f L^2_S}{k_f \phi_{Sc} f_{dy}}} \qquad (3.34.37a)$$

Table 3-38

FLEXURE, DIAGONAL TENSION, SHEAR AND ORTHOTROPY COEFFICIENTS FOR REINFORCED CONCRETE SLABS

($\theta' = 0.25$)

		Two-Way Isotropic			Two-Way Orthotropic*			
α	Support	k_f	k_{sc}	k_v	k_f(short)	k_{sc}	k_v	μ_e
1.0	Fixed	0.00300	2.350	0.0490	0.00300	2.350	0.0490	1.00
	Simple	0.00150	0.964	0.0490	0.00150	0.964	0.0490	
0.9	Fixed	0.00273	2.233	0.0466	0.00217	2.073	0.0450	1.70
	Simple	0.00136	0.916	0.0466	0.00108	0.850	0.0450	
0.8	Fixed	0.00249	2.115	0.0441	0.00173	1.937	0.0404	2.69
	Simple	0.00124	0.868	0.0441	0.00087	0.795	0.0404	
0.7	Fixed	0.00227	1.998	0.0417	0.00147	1.862	0.0388	4.13
	Simple	0.00113	0.819	0.0417	0.00074	0.764	0.0388	
0.6	Fixed	0.00207	1.880	0.0392	0.00131	1.818	0.0379	6.34
	Simple	0.00103	0.771	0.0392	0.00065	0.746	0.0379	
0.5	Fixed	0.00188	1.763	0.0368	0.00120	1.792	0.0374	10.00
	Simple	0.00094	0.723	0.0368	0.00060	0.735	0.0374	
One-Way Fixed		0.00100	1.765	0.0368				
One-Way Simple		0.00050	0.725	0.0368				

* Listed values of k_f, k_{sc}, and k_v for the orthotropic slab were obtained by assuming that $\mu = \mu_e$.

By assuming that the d= 0.9 D assumption used for the one-way slab cost studies can also be extended to the two-way slab, Equation 3.34.37a becomes

$$D = \frac{L_S}{0.9} \sqrt{\frac{q_f}{k_f \phi_{Sc} f_{dy}}} \qquad (3.34.37b)$$

Next, Equations 3.34.32 and 3.34.34 (with $\phi_v = 0$) can be solved simultaneously to obtain an expression for ϕ_{Sc} with $q_f = q_{sc}$

$$\phi_{Sc} = \left[\frac{1}{(2+\theta)} \quad \frac{k_{sc}}{k_f} \quad \frac{1}{f_{dy}} \right]^2 \quad f'_c \qquad (3.34.38)$$

Finally, values of ϕ_{Sc} satisfying Equation 3.34.38 can be substituted into Equation 3.34.37b. The resulting expression for D requires that $q_f = q_{sc}$, and is subsequently used in all two-way slab cost terms. The general cost factor for the concrete in a two-way slab is identical with the concrete cost factor for the one-way slab. However, the expressions for slab depth D will be dependent upon the type of slab which is considered.

$$C_c = \frac{X_c D}{12} \qquad (3.33.30a)$$

(2) Moment Steel Reinforcement

The cost factor for reinforcing steel, per square foot of fixed-end one-way reinforced slab, is supplied by Equation 3.33.30b. As explained in the derivation of this equation, trial layouts for reinforcing steel in one-way slabs were examined in order to develop a relationship between ϕ_c(max.) and ϕ_c(average). This same form of reasoning is applicable to two-way reinforced slabs, since the cost of main reinforcing steel can be treated as the linear sum of the cost of the reinforcement in each of the two directions. Writing Equation 3.33.30b for the L_S and L_L directions, and recalling that $\phi_S = \phi_L$ and $\alpha = \frac{L_S}{L_L}$, the following cost expressions are obtained.

One-Way $$C_s = X_s \left[1.33 + 0.278 \frac{f_{dy}}{f'_c} \quad \frac{1}{L_s} \right] \quad \frac{\phi_c D}{1333} \qquad (3.34.39a)$$

Isotropic, two-way,
$$C_s = X_s \left[2.66 + 0.278 \frac{f_{dy}}{f'_c} \frac{1}{L_S} (1+\alpha)\right] \frac{\phi_{Sc} D}{1333} \quad (3.34.39b)$$

Orthotropic, two-way
$$C_s = X_s \left[1.33\left(1+\frac{1}{\mu}\right) + 0.278 \frac{f_{dy}}{f'_c} \frac{1}{L_S} \left(1+\frac{\alpha}{\mu}\right)\right] \frac{\phi_{Sc} D}{1333} \quad (3.34.39c)$$

(3) Diagonal Tension Reinforcement

The expression for the cost of diagonal tension reinforcement in a unit area of one-way reinforced, fixed-edge slab is supplied as Equation 3.33.30c. This cost term is applicable only if web reinforcement is provided, and must be omitted for slabs without stirrups. The expression could readily be generalized, through the introduction of Equations 3.34.10b and 3.34.26b, to express the cost of stirrup steel in fixed-edge, two-way reinforced, isotropic and orthotropic slabs. However, the feasibility of installing stirrup steel in a two-way reinforced slab is questioned, particularly if stirrups are to be specified in two directions. For this reason, the cost studies of two-way slabs will assume that web steel will not be provided . Thus $C_v = 0$ for all cases studied. The resistance of the two-way slab to a diagonal tension mode of failure is expressed by Equation 3.34.10a for the isotropic slab and by Equation 3.34.26a for the orthotropic slab. Although $C_v = 0$, the constraint that $q_f = q_{sc}$ must be retained in the cost equations. This is accomplished by obtaining expressions for ϕ_c or ϕ_{Sc} when $q_f = q_{Sc}$, and substituting these into the appropriate expressions for slab depth, D.

(4) Temperature Reinforcement

Temperature reinforcement in slabs is required for one-way slabs only. The cost of temperature reinforcing steel in a square foot of slab, however, can be expressed in general terms. Assuming $d=0.9D$ and $\phi_{te} = 0.1$, Equation 3.33.30d expresses the cost of temperature steel for one-way reinforced slabs.

$$C_{st} = \frac{X_s D}{12,000} \quad (3.33.30d)$$

(5) Form Work

The general expression for the forming costs of one-way slabs is valid for all types of concrete slabs.

Overhead Slab $\qquad C_f = k_f = X_f + 0.012\,D \qquad (3.33.30e)$

Ground Level Slab $\qquad C_f = X_f \qquad (3.33.30f)$

Appropriate expressions for D, derived for one-way slabs or for osotropic or orthotropic slabs, can be inserted in Equation 3.33.30e or 3.33.30f. By virtue of their derivation, these expressions for D will include the constraint that $q_f = q_{sc}$.

(6) Total Cost

The total cost per square foot of slab, C_t, can now be obtained as the sum of the individual cost terms. The following equations apply to overhead slabs with fixed edge support.

(a) One-way slab

See Equations 3.33.33 and 3.33.34.

(b) Two-way isotropic (no web steel)

$$C_t = \left[\frac{X_c D}{12}\right] + \left[\frac{X_s \phi_{Sc}}{1333} D \left\{ 2.66 + \frac{0.278}{L_S} \frac{f_{dy}}{f'_c} (1 + \alpha)\right\}\right] + X_f + 0.012\,D \qquad (3.34.40)$$

where $\quad D = \dfrac{L_S}{0.9} \sqrt{\dfrac{q_f}{k_f \phi_{Sc} f_{dy}}}$

and $\quad \phi_{Sc} = \left[\dfrac{k_{sc}}{2.25\, k_f f_{dy}}\right]^2 f'_c$ when $q_f = q_{sc}$ and $\theta' = 0.25$

hence $\quad D = \dfrac{2.50\, L_S}{k_{sc}} \cdot \sqrt{q_f k_f \dfrac{f_{dy}}{f'_c}}$ when $q_f = q_{sc}$

(c) Two-way orthotropic (no web steel)

$$C_t = \left[\frac{X_c D}{12}\right] + \left[\frac{X_s \phi_{Sc} D}{1333} \left\{1.33\left(1 + \frac{1}{\mu}\right) + \frac{0.278}{L_S} \frac{f_{dy}}{f'_c}\left(1 + \frac{\alpha}{\mu}\right)\right\}\right] + \left[X_f + 0.120\,D\right] \qquad (3.34.41)$$

where D and ϕ_{Sc} are as defined for Equation 3.34.41, and

$$\mu = \mu_e = \frac{3-2\alpha^2}{\alpha^2}$$

(d) Minimum In-Place Costs

Minimum in-place costs for one-way reinforced concrete overhead slabs with fixed end supports are supplied in Table 3-15. These are computed from Equations 3.33.33 and 3.33.34, which evaluate the in-place costs of one-way slabs, with and without web reinforcement. Certain restrictions are placed on permissible values of ϕ_c, ϕ_v, and minimum slab depth, D. The cost equations assume that $\theta' = 0.25$, and include the constraint that $q_f = q_{sc}$. Minimum cost solutions, obtained through use of an optimization computer program with ϕ_v and f'_c treated as variables within established limits, are supplied for specific clear-span lengths and for several strengths of reinforcing steel. Alternatively, designs for one-way slabs can be prepared with the aid of Tables 3-12 to 3-14, inclusive. In-place costs can then be computed with the aid of Tables 2-7 to 2-9, inclusive, and Equations 3.33.30. All one-way slab designs, whether developed from the basic design tables or obtained directly from Table 3-15, must be checked for their resistance to a "pure" shear failure mode:

Tables 3-39 and 3-40 supply minimum in-place cost designs, based on Equations 3.34.40 and 3.34.41, for fixed-end, two-way reinforced, isotropic and orthotropic overhead slabs. Thus, they are comparable to Table 3-15 for one-way slabs. However the tabulated minimum - cost solutions assume that web reinforcement will not be provided for two-way slabs. As a consequence, very large slab depths are indicated in Table 3-39 and 3-40 for certain combinations of long spans and large unit loadings. For these cases, the diagonal tension resistance of the section controls the design of the slab. If such combinations of span and loading must actually be contemplated in a practical design situation, the use of some type of web reinforcement should definitely be considered.

Tables 3-39 and 3-40 include the assumptions that $q_f = q_{sc}$ and $\theta \doteq 0.25$. Values of ϕ_{Sc} must lie within specified minimum and maximum limits, and the total slab thickness must at least equal a specified minimum

value. Repetitive solutions, assuming finite levels of f'_c and applying these to specific values of f_{dy}, α, and L_S, are used to obtain the tabulated minimum-cost solutions. For the orthotropic slab (Table 3-40), it is assumed that values of μ equal to μ_e (see Reference 39) will be associated with minimum slab costs. Limited investigations during this study also suggest that the total cost of the orthotropic slab is rather insensitive to the choice of μ when the constraint that $q_f = q_{sc}$ is included in the cost equation.

Alternatively, if so desired, designs for isotropic or orthotropic slabs can be developed with the aid of Tables 3-16 to 3-37, inclusive. The in-place costs of such slabs can then be calculated, using cost data from Tables 2-7 to 2-9, inclusive, and applying cost equations 3.33.30a, 3.34.39b or 3.34.39c, and 3.33.30e or 3.33.30f. The cost of web reinforcement, if provided, can be including by modifying Equation 3.33.30c for the particular case being studied. As with the one-way slab, all designs for two-way slabs must be checked to ensure that their resistance is adequate in "pure" shear.

TABLE 3-39

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9$ D)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
7.0	1.0	10.	44000.	3000.	0.25	4.5	1.62
			52000.	3000.	0.25	4.5	1.63
			60000.	3500.	0.25	4.5	1.64
			75000.	4000.	0.25	4.5	1.69
		25.	44000.	6000.	0.38	5.5	1.97
			52000.	3000.	0.25	6.2	1.92
			60000.	3500.	0.25	5.8	1.86
			75000.	4000.	0.25	5.2	1.81
		50.	44000.	6000.	0.38	7.8	2.43
			52000.	3000.	0.25	8.8	2.35
			60000.	3500.	0.25	8.2	2.27
			75000.	4000.	0.25	7.3	2.20
		75.	44000.	6000.	0.38	9.6	2.78
			52000.	3000.	0.25	10.8	2.68
			60000.	3500.	0.25	10.0	2.58
			75000.	4000.	0.25	9.0	2.50
		100.	44000.	6000.	0.38	11.0	3.07
			52000.	3000.	0.25	12.5	2.96
			60000.	3500.	0.25	11.6	2.85
			75000.	4000.	0.25	10.4	2.75
		150.	44000.	6000.	0.38	13.5	3.56
			52000.	3000.	0.25	15.3	3.42
			60000.	3500.	0.25	14.2	3.29
			75000.	4000.	0.25	12.7	3.17
		200.	44000.	6000.	0.38	15.6	3.98
			52000.	3000.	0.25	17.6	3.82
			60000.	3500.	0.25	16.4	3.66
			75000.	4000.	0.25	14.7	3.52
		250.	44000.	6000.	0.38	17.5	4.34
			52000.	3000.	0.25	19.7	4.16
			60000.	3500.	0.25	18.3	3.99
			75000.	4000.	0.25	16.4	3.83
		300.	44000.	6000.	0.38	19.1	4.67
			52000.	3000.	0.25	21.6	4.48
			60000.	3500.	0.25	20.1	4.29
			75000.	4000.	0.25	18.0	4.12
		350.	44000.	6000.	0.38	20.7	4.98
			52000.	3000.	0.25	23.3	4.76
			60000.	3500.	0.25	21.7	4.56
			75000.	4000.	0.25	19.4	4.38

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9$ D)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D In.	Ct $/sq ft
7.0	0.9	10.	44000.	3000.	0.25	4.5	1.61
			52000.	3000.	0.25	4.5	1.63
			60000.	3000.	0.25	4.5	1.64
			75000.	4000.	0.25	4.5	1.69
		25.	44000.	6000.	0.41	5.5	2.01
			52000.	3000.	0.25	6.5	1.96
			60000.	3000.	0.25	6.1	1.90
			75000.	4000.	0.25	5.4	1.85
		50.	44000.	6000.	0.41	7.8	2.48
			52000.	3000.	0.25	9.2	2.41
			60000.	3000.	0.25	8.6	2.33
			75000.	4000.	0.25	7.7	2.26
		75.	44000.	6000.	0.41	9.6	2.84
			52000.	3000.	0.25	11.3	2.75
			60000.	3000.	0.25	10.5	2.65
			75000.	4000.	0.25	9.4	2.56
		100.	44000.	6000.	0.41	11.1	3.14
			52000.	3000.	0.25	13.1	3.04
			60000.	3000.	0.25	12.2	2.93
			75000.	4000.	0.25	10.9	2.83
		150.	44000.	6000.	0.41	13.6	3.65
			52000.	3000.	0.25	16.0	3.53
			60000.	3000.	0.25	14.9	3.39
			75000.	4000.	0.25	13.3	3.26
		200.	44000.	6000.	0.41	15.7	4.08
			52000.	3000.	0.25	18.5	3.94
			60000.	3000.	0.25	17.2	3.78
			75000.	4000.	0.25	15.4	3.63
		250.	44000.	6000.	0.41	17.5	4.46
			52000.	3000.	0.25	20.6	4.30
			60000.	3000.	0.25	19.2	4.12
			75000.	4000.	0.25	17.2	3.96
		300.	44000.	6000.	0.41	19.2	4.80
			52000.	3000.	0.25	22.6	4.63
			60000.	3000.	0.25	21.1	4.43
			75000.	4000.	0.25	18.8	4.25
		350.	44000.	6000.	0.41	20.7	5.11
			52000.	3000.	0.25	24.4	4.93
			60000.	3000.	0.25	22.7	4.71
			75000.	4000.	0.25	20.3	4.52

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($.0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\,D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
7.0	0.8	10.	44000.	2500.	0.25	4.7	1.64
			52000.	3000.	0.25	4.5	1.62
			60000.	3000.	0.25	4.5	1.63
			75000.	3500.	0.25	4.5	1.68
		25.	44000.	5500.	0.40	5.8	2.05
			52000.	3000.	0.25	6.8	2.01
			60000.	3000.	0.25	6.4	1.95
			75000.	3500.	0.25	5.7	1.89
		50.	44000.	5500.	0.40	8.3	2.53
			52000.	3000.	0.25	9.7	2.47
			60000.	3000.	0.25	9.0	2.39
			75000.	3500.	0.25	8.0	2.31
		75.	44000.	5500.	0.40	10.1	2.91
			52000.	3000.	0.25	11.8	2.83
			60000.	3000.	0.25	11.0	2.73
			75000.	3500.	0.25	9.9	2.63
		100.	44000.	5500.	0.40	11.7	3.22
			52000.	3000.	0.25	13.7	3.13
			60000.	3000.	0.25	12.7	3.01
			75000.	3500.	0.25	11.4	2.90
		150.	44000.	5500.	0.40	14.3	3.75
			52000.	3000.	0.25	16.7	3.64
			60000.	3000.	0.25	15.6	3.49
			75000.	3500.	0.25	13.9	3.36
		200.	44000.	5500.	0.40	16.5	4.19
			52000.	3000.	0.25	19.3	4.06
			60000.	3000.	0.25	18.0	3.89
			75000.	3500.	0.25	16.1	3.74
		250.	44000.	5500.	0.40	18.5	4.58
			52000.	3000.	0.25	21.6	4.44
			60000.	3000.	0.25	20.1	4.25
			75000.	3500.	0.25	18.0	4.08
		300.	44000.	5500.	0.40	20.2	4.93
			52000.	3000.	0.25	23.7	4.78
			60000.	3000.	0.25	22.0	4.57
			75000.	3500.	0.25	19.7	4.38
		350.	44000.	5500.	0.40	21.8	5.26
			52000.	3000.	0.25	25.6	5.09
			60000.	3000.	0.25	23.8	4.87
			75000.	3500.	0.25	21.3	4.66

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
7.0	0.7	10.	44000.	3500.	0.28	4.7	1.66
			52000.	3000.	0.25	4.5	1.62
			60000.	3000.	0.25	4.5	1.63
			75000.	3500.	0.25	4.5	1.67
		25.	44000.	5500.	0.43	5.9	2.09
			52000.	3000.	0.25	7.2	2.05
			60000.	3000.	0.25	6.7	1.99
			75000.	3500.	0.25	6.0	1.93
		50.	44000.	5500.	0.43	8.3	2.59
			52000.	3000.	0.25	10.1	2.54
			60000.	3000.	0.25	9.4	2.45
			75000.	3500.	0.25	8.4	2.37
		75.	44000.	5500.	0.43	10.2	2.98
			52000.	3000.	0.25	12.4	2.91
			60000.	3000.	0.25	11.5	2.80
			75000.	3500.	0.25	10.3	2.70
		100.	44000.	5500.	0.43	11.8	3.30
			52000.	3000.	0.25	14.3	3.22
			60000.	3000.	0.25	13.3	3.10
			75000.	3500.	0.25	11.9	2.98
		150.	44000.	5500.	0.43	14.5	3.85
			52000.	3000.	0.25	17.5	3.75
			60000.	3000.	0.25	16.3	3.59
			75000.	3500.	0.25	14.6	3.45
		200.	44000.	5500.	0.43	16.7	4.31
			52000.	3000.	0.25	20.2	4.19
			60000.	3000.	0.25	18.9	4.01
			75000.	3500.	0.25	16.9	3.85
		250.	44000.	5500.	0.43	18.7	4.71
			52000.	3000.	0.25	22.6	4.58
			60000.	3000.	0.25	21.1	4.38
			75000.	3500.	0.25	18.9	4.20
		300.	44000.	5500.	0.43	20.4	5.08
			52000.	3000.	0.25	24.8	4.94
			60000.	3000.	0.25	23.1	4.72
			75000.	3500.	0.25	20.6	4.52
		350.	44000.	5500.	0.43	22.1	5.41
			52000.	3000.	0.25	26.8	5.26
			60000.	3000.	0.25	24.9	5.03
			75000.	3500.	0.25	22.3	4.81

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(.0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
7.0	0.6	10.	44000.	3500.	0.29	4.7	1.69
			52000.	3000.	0.25	4.7	1.65
			60000.	3000.	0.25	4.5	1.62
			75000.	3500.	0.25	4.5	1.67
		25.	44000.	5000.	0.42	6.3	2.14
			52000.	6000.	0.36	6.2	2.09
			60000.	3000.	0.25	7.0	2.03
			75000.	3500.	0.25	6.2	1.97
		50.	44000.	5000.	0.42	8.9	2.66
			52000.	6000.	0.36	8.8	2.60
			60000.	3000.	0.25	9.9	2.51
			75000.	3500.	0.25	8.8	2.42
		75.	44000.	5000.	0.42	10.9	3.05
			52000.	6000.	0.36	10.8	2.98
			60000.	3000.	0.25	12.1	2.88
			75000.	3500.	0.25	10.8	2.77
		100.	44000.	5000.	0.42	12.6	3.39
			52000.	6000.	0.36	12.5	3.31
			60000.	3000.	0.25	14.0	3.18
			75000.	3500.	0.25	12.5	3.06
		150.	44000.	5000.	0.42	15.4	3.96
			52000.	6000.	0.36	15.3	3.85
			60000.	3000.	0.25	17.1	3.70
			75000.	3500.	0.25	15.3	3.55
		200.	44000.	5000.	0.42	17.8	4.43
			52000.	6000.	0.36	17.6	4.31
			60000.	3000.	0.25	19.7	4.14
			75000.	3500.	0.25	17.7	3.97
		250.	44000.	5000.	0.42	19.9	4.85
			52000.	6000.	0.36	19.7	4.72
			60000.	3000.	0.25	22.1	4.52
			75000.	3500.	0.25	19.7	4.33
		300.	44000.	5000.	0.42	21.8	5.23
			52000.	6000.	0.36	21.6	5.09
			60000.	3000.	0.25	24.2	4.87
			75000.	3500.	0.25	21.6	4.66
		350.	44000.	5000.	0.42	23.5	5.58
			52000.	6000.	0.36	23.3	5.42
			60000.	3000.	0.25	26.1	5.19
			75000.	3500.	0.25	23.4	4.97

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\,D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
10.5	1.0	10.	44000.	2500.	0.25	6.4	1.89
			52000.	2500.	0.25	5.9	1.82
			60000.	2500.	0.25	5.5	1.77
			75000.	3000.	0.25	4.9	1.72
		25.	44000.	2500.	0.25	10.2	2.47
			52000.	2500.	0.25	9.3	2.37
			60000.	2500.	0.25	8.7	2.29
			75000.	3000.	0.25	7.8	2.21
		50.	44000.	2500.	0.25	14.4	3.13
			52000.	2500.	0.25	13.2	2.99
			60000.	2500.	0.25	12.3	2.87
			75000.	3000.	0.25	11.0	2.76
		75.	44000.	2500.	0.25	17.6	3.64
			52000.	2500.	0.25	16.2	3.46
			60000.	2500.	0.25	15.1	3.32
			75000.	3000.	0.25	13.5	3.19
		100.	44000.	2500.	0.25	20.3	4.07
			52000.	2500.	0.25	18.7	3.86
			60000.	2500.	0.25	17.4	3.70
			75000.	3000.	0.25	15.6	3.54
		150.	44000.	2500.	0.25	24.9	4.78
			52000.	2500.	0.25	22.9	4.53
			60000.	2500.	0.25	21.3	4.33
			75000.	3000.	0.25	19.1	4.14
		200.	44000.	2500.	0.25	28.7	5.39
			52000.	2500.	0.25	26.4	5.09
			60000.	2500.	0.25	24.6	4.86
			75000.	3000.	0.25	22.0	4.64
		250.	44000.	2500.	0.25	32.1	5.92
			52000.	2500.	0.25	29.5	5.59
			60000.	2500.	0.25	27.5	5.33
			75000.	3000.	0.25	24.6	5.09
		300.	44000.	2500.	0.25	35.2	6.40
			52000.	2500.	0.25	32.4	6.04
			60000.	2500.	0.25	30.1	5.76
			75000.	3000.	0.25	26.9	5.49
		350.	44000.	2500.	0.25	38.0	6.84
			52000.	2500.	0.25	35.0	6.45
			60000.	2500.	0.25	32.5	6.15
			75000.	3000.	0.25	29.1	5.86

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($.0.25 \le \phi_{sc} = \phi_{Lc} \le 2.00$, $\phi_v = 0$, $2000 \le f'_c \le 6000$, $d = 0.9\,D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
10.5	0.9	10.	44000.	5500.	0.38	5.5	1.93
			52000.	2500.	0.25	6.2	1.86
			60000.	2500.	0.25	5.8	1.81
			75000.	3000.	0.25	5.2	1.76
		25.	44000.	5500.	0.38	8.7	2.54
			52000.	2500.	0.25	9.8	2.43
			60000.	2500.	0.25	9.1	2.35
			75000.	3000.	0.25	8.2	2.27
		50.	44000.	5500.	0.38	12.3	3.23
			52000.	2500.	0.25	13.8	3.08
			60000.	2500.	0.25	12.9	2.96
			75000.	3000.	0.25	11.5	2.84
		75.	44000.	5500.	0.38	15.0	3.75
			52000.	2500.	0.25	17.0	3.57
			60000.	2500.	0.25	15.8	3.42
			75000.	3000.	0.25	14.1	3.28
		100.	44000.	5500.	0.38	17.4	4.20
			52000.	2500.	0.25	19.6	3.99
			60000.	2500.	0.25	18.2	3.81
			75000.	3000.	0.25	16.3	3.65
		150.	44000.	5500.	0.38	21.3	4.94
			52000.	2500.	0.25	24.0	4.68
			60000.	2500.	0.25	22.3	4.47
			75000.	3000.	0.25	20.0	4.28
		200.	44000.	5500.	0.38	24.6	5.57
			52000.	2500.	0.25	27.7	5.27
			60000.	2500.	0.25	25.8	5.03
			75000.	3000.	0.25	23.1	4.80
		250.	44000.	5500.	0.38	27.5	6.12
			52000.	2500.	0.25	31.0	5.79
			60000.	2500.	0.25	28.8	5.52
			75000.	3000.	0.25	25.8	5.26
		300.	44000.	5500.	0.38	30.1	6.63
			52000.	2500.	0.25	33.9	6.26
			60000.	2500.	0.25	31.6	5.96
			75000.	3000.	0.25	28.2	5.68
		350.	44000.	5500.	0.38	32.5	7.09
			52000.	2500.	0.25	36.6	6.69
			60000.	2500.	0.25	34.1	6.37
			75000.	3000.	0.25	30.5	6.07

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
10.5	0.8	10.	44000.	5500.	0.40	5.5	1.96
			52000.	2500.	0.25	6.5	1.90
			60000.	2500.	0.25	6.0	1.85
			75000.	3000.	0.25	5.4	1.79
		25.	44000.	5500.	0.40	8.8	2.59
			52000.	2500.	0.25	10.3	2.50
			60000.	2500.	0.25	9.5	2.41
			75000.	3000.	0.25	8.5	2.32
		50.	44000.	5500.	0.40	12.4	3.30
			52000.	2500.	0.25	14.5	3.17
			60000.	2500.	0.25	13.5	3.04
			75000.	3000.	0.25	12.1	2.92
		75.	44000.	5500.	0.40	15.2	3.85
			52000.	2500.	0.25	17.8	3.68
			60000.	2500.	0.25	16.5	3.53
			75000.	3000.	0.25	14.8	3.38
		100.	44000.	5500.	0.40	17.5	4.31
			52000.	2500.	0.25	20.5	4.12
			60000.	2500.	0.25	19.1	3.93
			75000.	3000.	0.25	17.1	3.77
		150.	44000.	5500.	0.40	21.5	5.08
			52000.	2500.	0.25	25.1	4.84
			60000.	2500.	0.25	23.4	4.62
			75000.	3000.	0.25	20.9	4.41
		200.	44000.	5500.	0.40	24.8	5.73
			52000.	2500.	0.25	29.0	5.46
			60000.	2500.	0.25	27.0	5.20
			75000.	3000.	0.25	24.1	4.96
		250.	44000.	5500.	0.40	27.7	6.30
			52000.	2500.	0.25	32.4	6.00
			60000.	2500.	0.25	30.2	5.71
			75000.	3000.	0.25	27.0	5.44
		300.	44000.	5500.	0.40	30.3	6.82
			52000.	2500.	0.25	35.5	6.48
			60000.	2500.	0.25	33.1	6.17
			75000.	3000.	0.25	29.6	5.88
		350.	44000.	5500.	0.40	32.8	7.30
			52000.	2500.	0.25	38.4	6.93
			60000.	2500.	0.25	35.7	6.59
			75000.	3000.	0.25	31.9	6.28

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS, WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
10.5	0.7	10.	44000.	5000.	0.40	5.9	2.00
			52000.	2500.	0.25	6.8	1.95
			60000.	2500.	0.25	6.3	1.89
			75000.	3000.	0.25	5.7	1.83
		25.	44000.	5000.	0.40	9.3	2.66
			52000.	2500.	0.25	10.7	2.57
			60000.	2500.	0.25	10.0	2.47
			75000.	3000.	0.25	8.9	2.38
		50.	44000.	5000.	0.40	13.1	3.39
			52000.	2500.	0.25	15.2	3.26
			60000.	2500.	0.25	14.1	3.13
			75000.	3000.	0.25	12.6	3.00
		75.	44000.	5000.	0.40	16.1	3.95
			52000.	2500.	0.25	18.6	3.80
			60000.	2500.	0.25	17.3	3.63
			75000.	3000.	0.25	15.5	3.48
		100.	44000.	5000.	0.40	18.6	4.43
			52000.	2500.	0.25	21.5	4.25
			60000.	2500.	0.25	20.0	4.06
			75000.	3000.	0.25	17.9	3.88
		150.	44000.	5000.	0.40	22.7	5.23
			52000.	2500.	0.25	26.3	5.01
			60000.	2500.	0.25	24.5	4.78
			75000.	3000.	0.25	21.9	4.56
		200.	44000.	5000.	0.40	26.3	5.90
			52000.	2500.	0.25	30.4	5.65
			60000.	2500.	0.25	28.3	5.38
			75000.	3000.	0.25	25.3	5.13
		250.	44000.	5000.	0.40	29.4	6.49
			52000.	2500.	0.25	34.0	6.21
			60000.	2500.	0.25	31.6	5.91
			75000.	3000.	0.25	28.3	5.63
		300.	44000.	5000.	0.40	32.2	7.03
			52000.	2500.	0.25	37.2	6.72
			60000.	2500.	0.25	34.6	6.39
			75000.	3000.	0.25	31.0	6.08
		350.	44000.	5000.	0.40	34.7	7.52
			52000.	2500.	0.25	40.2	7.19
			60000.	2500.	0.25	37.4	6.83
			75000.	3000.	0.25	33.5	6.50

TALBE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(.0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
10.5	0.6	10.	44000.	5000.	0.42	6.0	2.04
			52000.	2500.	0.25	7.1	1.99
			60000.	2500.	0.25	6.6	1.93
			75000.	3000.	0.25	5.9	1.87
		25.	44000.	5000.	0.42	9.4	2.72
			52000.	2500.	0.25	11.2	2.64
			60000.	2500.	0.25	10.5	2.54
			75000.	3000.	0.25	9.4	2.44
		50.	44000.	5000.	0.42	13.3	3.48
			52000.	2500.	0.25	15.9	3.36
			60000.	2500.	0.25	14.8	3.22
			75000.	3000.	0.25	13.2	3.09
		75.	44000.	5000.	0.42	16.3	4.07
			52000.	2500.	0.25	19.5	3.92
			60000.	2500.	0.25	18.1	3.75
			75000.	3000.	0.25	16.2	3.59
		100.	44000.	5000.	0.42	18.8	4.56
			52000.	2500.	0.25	22.5	4.39
			60000.	2500.	0.25	20.9	4.19
			75000.	3000.	0.25	18.7	4.00
		150.	44000.	5000.	0.42	23.1	5.39
			52000.	2500.	0.25	27.5	5.18
			60000.	2500.	0.25	25.6	4.94
			75000.	3000.	0.25	22.9	4.71
		200.	44000.	5000.	0.42	26.7	6.08
			52000.	2500.	0.25	31.8	5.85
			60000.	2500.	0.25	29.6	5.56
			75000.	3000.	0.25	26.5	5.30
		250.	44000.	5000.	0.42	29.8	6.70
			52000.	2500.	0.25	35.6	6.43
			60000.	2500.	0.25	33.1	6.12
			75000.	3000.	0.25	29.6	5.82
		300.	44000.	5000.	0.42	32.6	7.25
			52000.	2500.	0.25	39.0	6.96
			60000.	2500.	0.25	36.3	6.61
			75000.	3000.	0.25	32.4	6.29
		350.	44000.	5000.	0.42	35.3	7.76
			52000.	2500.	0.25	42.1	7.45
			60000.	2500.	0.25	39.2	7.07
			75000.	3000.	0.25	35.0	6.73

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(.0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
14.0	1.0	10.	44000.	2000.	0.25	8.6	2.19
			52000.	2000.	0.25	7.9	2.10
			60000.	2500.	0.25	7.3	2.03
			75000.	3000.	0.25	6.6	1.97
		25.	44000.	2000.	0.25	13.5	2.95
			52000.	2000.	0.25	12.5	2.81
			60000.	2500.	0.25	11.6	2.70
			75000.	3000.	0.25	10.4	2.60
		50.	44000.	2000.	0.25	19.1	3.81
			52000.	2000.	0.25	17.6	3.61
			60000.	2500.	0.25	16.4	3.46
			75000.	3000.	0.25	14.7	3.31
		75.	44000.	2000.	0.25	23.5	4.46
			52000.	2000.	0.25	21.6	4.23
			60000.	2500.	0.25	20.1	4.04
			75000.	3000.	0.25	18.0	3.86
		100.	44000.	2000.	0.25	27.1	5.02
			52000.	2000.	0.25	24.9	4.75
			60000.	2500.	0.25	23.2	4.53
			75000.	3000.	0.25	20.7	4.32
		150.	44000.	2000.	0.25	33.2	5.95
			52000.	2000.	0.25	30.5	5.61
			60000.	2500.	0.25	28.4	5.34
			75000.	3000.	0.25	25.4	5.09
		200.	44000.	2000.	0.25	38.3	6.73
			52000.	2000.	0.25	35.2	6.35
			60000.	2500.	0.25	32.8	6.04
			75000.	3000.	0.25	29.3	5.74
		250.	44000.	2000.	0.25	42.8	7.42
			52000.	2000.	0.25	39.4	6.99
			60000.	2500.	0.25	36.7	6.64
			75000.	3000.	0.25	32.8	6.32
		300.	44000.	2000.	0.25	46.9	8.05
			52000.	2000.	0.25	43.1	7.58
			60000.	2500.	0.25	40.2	7.19
			75000.	3000.	0.25	35.9	6.84
		350.	44000.	2000.	0.25	50.7	8.62
			52000.	2000.	0.25	46.6	8.11
			60000.	2500.	0.25	43.4	7.70
			75000.	3000.	0.25	38.8	7.31

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($.0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = o, 2000 \leq f'_c \leq 6000,\ d = 0.9$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
14.0	0.9	10.	44000.	2000.	0.25	9.0	2.25
			52000.	2000.	0.25	8.3	2.16
			60000.	2500.	0.25	7.7	2.08
			75000.	2500.	0.25	6.9	2.01
		25.	44000.	2000.	0.25	14.2	3.04
			52000.	2000.	0.25	13.1	2.90
			60000.	2500.	0.25	12.2	2.78
			75000.	2500.	0.25	10.9	2.67
		50.	44000.	2000.	0.25	20.1	3.93
			52000.	2000.	0.25	18.5	3.73
			60000.	2500.	0.25	17.2	3.57
			75000.	2500.	0.25	15.4	3.41
		75.	44000.	2000.	0.25	24.6	4.62
			52000.	2000.	0.25	22.6	4.37
			60000.	2500.	0.25	21.1	4.17
			75000.	2500.	0.25	18.8	3.98
		100.	44000.	2000.	0.25	28.4	5.20
			52000.	2000.	0.25	26.1	4.91
			60000.	2500.	0.25	24.3	4.68
			75000.	2500.	0.25	21.7	4.46
		150.	44000.	2000.	0.25	34.8	6.17
			52000.	2000.	0.25	32.0	5.82
			60000.	2500.	0.25	29.8	5.54
			75000.	2500.	0.25	26.6	5.27
		200.	44000.	2000.	0.25	40.1	6.99
			52000.	2000.	0.25	36.9	6.58
			60000.	2500.	0.25	34.4	6.26
			75000.	2500.	0.25	30.7	5.95
		250.	44000.	2000.	0.25	44.9	7.71
			52000.	2000.	0.25	41.3	7.26
			60000.	2500.	0.25	38.4	6.90
			75000.	2500.	0.25	34.4	6.55
		300.	44000.	2000.	0.25	49.2	8.36
			52000.	2000.	0.25	45.2	7.86
			60000.	2500.	0.25	42.1	7.47
			75000.	2500.	0.25	37.7	7.09
		350.	44000.	2000.	0.25	53.1	8.96
			52000.	2000.	0.25	48.9	8.42
			60000.	2500.	0.25	45.5	8.00
			75000.	2500.	0.25	40.7	7.58

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\,D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
14.0	0.8	10.	44000.	2000.	0.25	9.4	2.30
			52000.	2000.	0.25	8.6	2.21
			60000.	2000.	0.25	8.0	2.13
			75000.	2500.	0.25	7.2	2.06
		25.	44000.	2000.	0.25	14.9	3.13
			52000.	2000.	0.25	13.7	2.98
			60000.	2000.	0.25	12.7	2.86
			75000.	2500.	0.25	11.4	2.75
		50.	44000.	2000.	0.25	21.0	4.07
			52000.	2000.	0.25	19.3	3.85
			60000.	2000.	0.25	18.0	3.68
			75000.	2500.	0.25	16.1	3.52
		75.	44000.	2000.	0.25	25.7	4.78
			52000.	2000.	0.25	23.7	4.52
			60000.	2000.	0.25	22.0	4.31
			75000.	2500.	0.25	19.7	4.11
		100.	44000.	2000.	0.25	29.7	5.38
			52000.	2000.	0.25	27.3	5.08
			60000.	2000.	0.25	25.5	4.84
			75000.	2500.	0.25	22.8	4.61
		150.	44000.	2000.	0.25	36.4	6.40
			52000.	2000.	0.25	33.5	6.03
			60000.	2000.	0.25	31.2	5.74
			75000.	2500.	0.25	27.9	5.45
		200.	44000.	2000.	0.25	42.0	7.25
			52000.	2000.	0.25	38.7	6.82
			60000.	2000.	0.25	36.0	6.49
			75000.	2500.	0.25	32.2	6.16
		250.	44000.	2000.	0.25	47.0	8.00
			52000.	2000.	0.25	43.2	7.52
			60000.	2000.	0.25	40.2	7.15
			75000.	2500.	0.25	36.0	6.78
		300.	44000.	2000.	0.25	51.5	8.68
			52000.	2000.	0.25	47.4	8.16
			60000.	2000.	0.25	44.1	7.75
			75000.	2500.	0.25	39.4	7.34
		350.	44000.	2000.	0.25	55.6	9.31
			52000.	2000.	0.25	51.2	8.74
			60000.	2000.	0.25	47.6	8.30
			75000.	2500.	0.25	42.6	7.86

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPTIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9$ D)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
14.0	0.7	10.	44000.	5000.	0.40	7.8	2.36
			52000.	2000.	0.25	9.1	2.26
			60000.	2000.	0.25	8.4	2.19
			75000.	2500.	0.25	7.5	2.11
		25.	44000.	5000.	0.40	12.4	3.22
			52000.	2000.	0.25	14.3	3.07
			60000.	2000.	0.25	13.3	2.94
			75000.	2500.	0.25	11.9	2.82
		50.	44000.	5000.	0.40	17.5	4.19
			52000.	2000.	0.25	20.2	3.98
			60000.	2000.	0.25	18.9	3.80
			75000.	2500.	0.25	16.9	3.63
		75.	44000.	5000.	0.40	21.4	4.93
			52000.	2000.	0.25	24.8	4.67
			60000.	2000.	0.25	23.1	4.46
			75000.	2500.	0.25	20.6	4.24
		100.	44000.	5000.	0.40	24.8	5.56
			52000.	2000.	0.25	28.6	5.26
			60000.	2000.	0.25	26.7	5.01
			75000.	2500.	0.25	23.8	4.76
		150.	44000.	5000.	0.40	30.3	6.61
			52000.	2000.	0.25	35.1	6.24
			60000.	2000.	0.25	32.6	5.94
			75000.	2500.	0.25	29.2	5.64
		200.	44000.	5000.	0.40	35.0	7.49
			52000.	2000.	0.25	40.5	7.07
			60000.	2000.	0.25	37.7	6.72
			75000.	2500.	0.25	33.7	6.37
		250.	44000.	5000.	0.40	39.1	8.27
			52000.	2000.	0.25	45.3	7.80
			60000.	2000.	0.25	42.1	7.41
			75000.	2500.	0.25	37.7	7.02
		300.	44000.	5000.	0.40	42.9	8.98
			52000.	2000.	0.25	49.6	8.46
			60000.	2000.	0.25	46.2	8.03
			75000.	2500.	0.25	41.3	7.61
		350.	44000.	5000.	0.40	46.3	9.63
			52000.	2000.	0.25	53.6	9.07
			60000.	2000.	0.25	49.9	8.60
			75000.	2500.	0.25	44.6	8.15

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED - END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(.0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
14.0	0.6	10.	44000.	5000.	0.42	7.9	2.41
			52000.	2000.	0.25	9.5	2.32
			60000.	2000.	0.25	8.8	2.24
			75000.	2500.	0.25	7.9	2.16
		25.	44000.	5000.	0.42	12.6	3.30
			52000.	2000.	0.25	15.0	3.16
			60000.	2000.	0.25	14.0	3.03
			75000.	2500.	0.25	12.5	2.90
		50.	44000.	5000.	0.42	17.8	4.31
			52000.	2000.	0.25	21.2	4.11
			60000.	2000.	0.25	19.7	3.92
			75000.	2500.	0.25	17.7	3.74
		75.	44000.	5000.	0.42	21.8	5.08
			52000.	2000.	0.25	26.0	4.83
			60000.	2000.	0.25	24.2	4.60
			75000.	2500.	0.25	21.6	4.38
		100.	44000.	5000.	0.42	25.1	5.73
			52000.	2000.	0.25	30.0	5.44
			60000.	2000.	0.25	27.9	5.18
			75000.	2500.	0.25	25.0	4.92
		150.	44000.	5000.	0.42	30.8	6.82
			52000.	2000.	0.25	36.7	6.47
			60000.	2000.	0.25	34.2	6.15
			75000.	2500.	0.25	30.6	5.83
		200.	44000.	5000.	0.42	35.5	7.73
			52000.	2000.	0.25	42.4	7.33
			60000.	2000.	0.25	39.5	6.96
			75000.	2500.	0.25	35.3	6.60
		250.	44000.	5000.	0.42	39.7	8.54
			52000.	2000.	0.25	47.4	8.09
			60000.	2000.	0.25	44.1	7.68
			75000.	2500.	0.25	39.5	7.27
		300.	44000.	5000.	0.42	43.5	9.28
			52000.	2000.	0.25	51.9	8.78
			60000.	2000.	0.25	48.4	8.33
			75000.	2500.	0.25	43.2	7.88
		350.	44000.	5000.	0.42	47.0	9.95
			52000.	2000.	0.25	56.1	9.41
			60000.	2000.	0.25	52.2	8.92
			75000.	2500.	0.25	46.7	8.44

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(.0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
17.5	1.0	10.	44000.	2000.	0.25	10.7	2.49
			52000.	2000.	0.25	9.8	2.38
			60000.	2000.	0.25	9.2	2.29
			75000.	2500.	0.25	8.2	2.21
		25.	44000.	2000.	0.25	16.9	3.42
			52000.	2000.	0.25	15.6	3.25
			60000.	2000.	0.25	14.5	3.11
			75000.	2500.	0.25	13.0	2.98
		50.	44000.	2000.	0.25	23.9	4.48
			52000.	2000.	0.25	22.0	4.23
			60000.	2000.	0.25	20.5	4.04
			75000.	2500.	0.25	18.3	3.85
		75.	44000.	2000.	0.25	29.3	5.28
			52000.	2000.	0.25	27.0	4.98
			60000.	2000.	0.25	25.1	4.75
			75000.	2500.	0.25	22.5	4.52
		100.	44000.	2000.	0.25	33.8	5.97
			52000.	2000.	0.25	31.1	5.62
			60000.	2000.	0.25	29.0	5.34
			75000.	2500.	0.25	25.9	5.08
		150.	44000.	2000.	0.25	41.5	7.11
			52000.	2000.	0.25	38.1	6.68
			60000.	2000.	0.25	35.5	6.35
			75000.	2500.	0.25	31.8	6.02
		200.	44000.	2000.	0.25	47.9	8.07
			52000.	2000.	0.25	44.0	7.58
			60000.	2000.	0.25	41.0	7.19
			75000.	2500.	0.25	36.7	6.82
		250.	44000.	2000.	0.25	53.5	8.92
			52000.	2000.	0.25	49.2	8.37
			60000.	2000.	0.25	45.8	7.94
			75000.	2500.	0.25	41.0	7.52
		300.	44000.	2000.	0.25	58.6	9.69
			52000.	2000.	0.25	53.9	9.08
			60000.	2000.	0.25	50.2	8.61
			75000.	2500.	0.25	44.9	8.15
		350.	44000.	2000.	0.25	63.3	10.39
			52000.	2000.	0.25	58.3	9.74
			60000.	2000.	0.25	54.2	9.23
			75000.	2500.	0.25	48.5	8.73

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
17.5	0.9	10.	44000.	2000.	0.25	11.2	2.56
			52000.	2000.	0.25	10.3	2.44
			60000.	2000.	0.25	9.6	2.35
			75000.	2500.	0.25	8.6	2.27
		25.	44000.	2000.	0.25	17.7	3.54
			52000.	2000.	0.25	16.3	3.35
			60000.	2000.	0.25	15.2	3.21
			75000.	2500.	0.25	13.6	3.07
		50.	44000.	2000.	0.25	25.1	4.64
			52000.	2000.	0.25	23.1	4.38
			60000.	2000.	0.25	21.5	4.17
			75000.	2500.	0.25	19.2	3.98
		75.	44000.	2000.	0.25	30.7	5.48
			52000.	2000.	0.25	28.3	5.16
			60000.	2000.	0.25	26.3	4.91
			75000.	2500.	0.25	23.5	4.67
		100.	44000.	2000.	0.25	35.5	6.19
			52000.	2000.	0.25	32.6	5.83
			60000.	2000.	0.25	30.4	5.54
			75000.	2500.	0.25	27.2	5.26
		150.	44000.	2000.	0.25	43.5	7.39
			52000.	2000.	0.25	40.0	6.94
			60000.	2000.	0.25	37.2	6.59
			75000.	2500.	0.25	33.3	6.25
		200.	44000.	2000.	0.25	50.2	8.39
			52000.	2000.	0.25	46.2	7.87
			60000.	2000.	0.25	43.0	7.47
			75000.	2500.	0.25	38.4	7.07
		250.	44000.	2000.	0.25	56.1	9.28
			52000.	2000.	0.25	51.6	8.70
			60000.	2000.	0.25	48.0	8.25
			75000.	2500.	0.25	43.0	7.81
		300.	44000.	2000.	0.25	61.5	10.08
			52000.	2000.	0.25	56.5	9.45
			60000.	2000.	0.25	52.6	8.95
			75000.	2500.	0.25	47.1	8.47
		350.	44000.	2000.	0.25	66.4	10.82
			52000.	2000.	0.25	61.1	10.13
			60000.	2000.	0.25	56.8	9.59
			75000.	2500.	0.25	50.8	9.08

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_s ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
17.5	0.8	10.	44000.	2000.	0.25	11.7	2.63
			52000.	2000.	0.25	10.8	2.51
			60000.	2000.	0.25	10.1	2.42
			75000.	2500.	0.25	9.0	2.32
		25.	44000.	2000.	0.25	18.6	3.65
			52000.	2000.	0.25	17.1	3.46
			60000.	2000.	0.25	15.9	3.31
			75000.	2500.	0.25	14.2	3.16
		50.	44000.	2000.	0.25	26.3	4.80
			52000.	2000.	0.25	24.2	4.53
			60000.	2000.	0.25	22.5	4.31
			75000.	2500.	0.25	20.1	4.11
		75.	44000.	2000.	0.25	32.2	5.68
			52000.	2000.	0.25	29.6	5.35
			60000.	2000.	0.25	27.6	5.08
			75000.	2500.	0.25	24.6	4.83
		100.	44000.	2000.	0.25	37.2	6.42
			52000.	2000.	0.25	34.2	6.04
			60000.	2000.	0.25	31.8	5.74
			75000.	2500.	0.25	28.5	5.44
		150.	44000.	2000.	0.25	45.5	7.67
			52000.	2000.	0.25	41.9	7.20
			60000.	2000.	0.25	39.0	6.83
			75000.	2500.	0.25	34.9	6.47
		200.	44000.	2000.	0.25	52.5	8.72
			52000.	2000.	0.25	48.3	8.17
			60000.	2000.	0.25	45.0	7.75
			75000.	2500.	0.25	40.2	7.34
		250.	44000.	2000.	0.25	58.7	9.65
			52000.	2000.	0.25	54.0	9.03
			60000.	2000.	0.25	50.3	8.56
			75000.	2500.	0.25	45.0	8.10
		300.	44000.	2000.	0.25	64.4	10.48
			52000.	2000.	0.25	59.2	9.81
			60000.	2000.	0.25	55.1	9.29
			75000.	2500.	0.25	49.3	8.79
		350.	44000.	2000.	0.25	69.5	11.25
			52000.	2000.	0.25	63.9	10.53
			60000.	2000.	0.25	59.5	9.96
			75000.	2500.	0.25	53.2	9.42

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9$ D)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
17.5	0.7	10.	44000.	2000.	0.25	12.3	2.71
			52000.	2000.	0.25	11.3	2.58
			60000.	2000.	0.25	10.5	2.48
			75000.	2500.	0.25	9.4	2.38
		25.	44000.	2000.	0.25	19.5	3.77
			52000.	2000.	0.25	17.9	3.57
			60000.	2000.	0.25	16.7	3.41
			75000.	2500.	0.25	14.9	3.26
		50.	44000.	2000.	0.25	27.5	4.97
			52000.	2000.	0.25	25.3	4.68
			60000.	2000.	0.25	23.6	4.46
			75000.	2500.	0.25	21.1	4.24
		75.	44000.	2000.	0.25	33.7	5.89
			52000.	2000.	0.25	31.0	5.54
			60000.	2000.	0.25	28.9	5.26
			75000.	2500.	0.25	25.8	5.00
		100.	44000.	2000.	0.25	38.9	6.67
			52000.	2000.	0.25	35.8	6.26
			60000.	2000.	0.25	33.3	5.94
			75000.	2500.	0.25	29.8	5.64
		150.	44000.	2000.	0.25	47.7	7.97
			52000.	2000.	0.25	43.8	7.47
			60000.	2000.	0.25	40.8	7.08
			75000.	2500.	0.25	36.5	6.71
		200.	44000.	2000.	0.25	55.0	9.06
			52000.	2000.	0.25	50.6	8.49
			60000.	2000.	0.25	47.1	8.04
			75000.	2500.	0.25	42.1	7.61
		250.	44000.	2000.	0.25	61.5	10.03
			52000.	2000.	0.25	56.6	9.39
			60000.	2000.	0.25	52.7	8.88
			75000.	2500.	0.25	47.1	8.40
		300.	44000.	2000.	0.25	67.4	10.90
			52000.	2000.	0.25	62.0	10.20
			60000.	2000.	0.25	57.7	9.65
			75000.	2500.	0.25	51.6	9.12
		350.	44000.	2000.	0.25	72.8	11.71
			52000.	2000.	0.25	67.0	10.94
			60000.	2000.	0.25	62.3	10.35
			75000.	2500.	0.25	55.8	9.78

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = o,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
17.5	0.6	10.	44000.	5000.	0.42	9.9	2.78
			52000.	2000.	0.25	11.9	2.65
			60000.	2000.	0.25	11.0	2.55
			75000.	2000.	0.25	9.9	2.45
		25.	44000.	5000.	0.42	15.7	3.89
			52000.	2000.	0.25	18.7	3.69
			60000.	2000.	0.25	17.4	3.52
			75000.	2000.	0.25	15.6	3.36
		50.	44000.	5000.	0.42	22.2	5.13
			52000.	2000.	0.25	26.5	4.85
			60000.	2000.	0.25	24.7	4.61
			75000.	2000.	0.25	22.1	4.38
		75.	44000.	5000.	0.42	27.2	6.09
			52000.	2000.	0.25	32.5	5.74
			60000.	2000.	0.25	30.2	5.45
			75000.	2000.	0.25	27.0	5.17
		100.	44000.	5000.	0.42	31.4	6.90
			52000.	2000.	0.25	37.5	6.49
			60000.	2000.	0.25	34.9	6.16
			75000.	2000.	0.25	31.2	5.83
		150.	44000.	5000.	0.42	38.5	8.25
			52000.	2000.	0.25	45.9	7.75
			60000.	2000.	0.25	42.7	7.34
			75000.	2000.	0.25	38.2	6.95
		200.	44000.	5000.	0.42	44.4	9.39
			52000.	2000.	0.25	53.0	8.81
			60000.	2000.	0.25	49.3	8.34
			75000.	2000.	0.25	44.1	7.89
		250.	44000.	5000.	0.42	49.7	10.39
			52000.	2000.	0.25	59.3	9.75
			60000.	2000.	0.25	55.2	9.22
			75000.	2000.	0.25	49.3	8.71
		300.	44000.	5000.	0.42	54.4	11.30
			52000.	2000.	0.25	64.9	10.60
			60000.	2000.	0.25	60.4	10.02
			75000.	2000.	0.25	54.1	9.46
		350.	44000.	5000.	0.42	58.8	12.13
			52000.	2000.	0.25	70.1	11.38
			60000.	2000.	0.25	65.3	10.75
			75000.	2000.	0.25	58.4	10.15

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(.0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
21.0	1.0	10.	44000.	2000.	0.25	12.8	2.79
			52000.	2000.	0.25	11.8	2.65
			60000.	2000.	0.25	11.0	2.55
			75000.	2500.	0.25	9.8	2.45
		25.	44000.	2000.	0.25	20.3	3.90
			52000.	2000.	0.25	18.7	3.68
			60000.	2000.	0.25	17.4	3.52
			75000.	2500.	0.25	15.6	3.36
		50.	44000.	2000.	0.25	28.7	5.15
			52000.	2000.	0.25	26.4	4.85
			60000.	2000.	0.25	24.6	4.61
			75000.	2500.	0.25	22.0	4.39
		75.	44000.	2000.	0.25	35.2	6.10
			52000.	2000.	0.25	32.4	5.74
			60000.	2000.	0.25	30.1	5.45
			75000.	2500.	0.25	26.9	5.17
		100.	44000.	2000.	0.25	40.6	6.91
			52000.	2000.	0.25	37.4	6.49
			60000.	2000.	0.25	34.8	6.16
			75000.	2500.	0.25	31.1	5.84
		150.	44000.	2000.	0.25	49.7	8.27
			52000.	2000.	0.25	45.8	7.75
			60000.	2000.	0.25	42.6	7.34
			75000.	2500.	0.25	38.1	6.95
		200.	44000.	2000.	0.25	57.4	9.41
			52000.	2000.	0.25	52.8	8.81
			60000.	2000.	0.25	49.2	8.34
			75000.	2500.	0.25	44.0	7.89
		250.	44000.	2000.	0.25	64.2	10.42
			52000.	2000.	0.25	59.1	9.75
			60000.	2000.	0.25	55.0	9.22
			75000.	2500.	0.25	49.2	8.72
		300.	44000.	2000.	0.25	70.4	11.33
			52000.	2000.	0.25	64.7	10.59
			60000.	2000.	0.25	60.2	10.02
			75000.	2500.	0.25	53.9	9.47
		350.	44000.	2000.	0.25	76.0	12.17
			52000.	2000.	0.25	69.9	11.37
			60000.	2000.	0.25	65.1	10.75
			75000.	2500.	0.25	58.2	10.16

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
21.0	0.9	10.	44000.	2000.	0.25	13.5	2.87
			52000.	2000.	0.25	12.4	2.73
			60000.	2000.	0.25	11.5	2.62
			75000.	2000.	0.25	10.3	2.52
		25.	44000.	2000.	0.25	21.3	4.03
			52000.	2000.	0.25	19.6	3.81
			60000.	2000.	0.25	18.2	3.63
			75000.	2000.	0.25	16.3	3.47
		50.	44000.	2000.	0.25	30.1	5.34
			52000.	2000.	0.25	27.7	5.02
			60000.	2000.	0.25	25.8	4.78
			75000.	2000.	0.25	23.1	4.54
		75.	44000.	2000.	0.25	36.9	6.34
			52000.	2000.	0.25	33.9	5.95
			60000.	2000.	0.25	31.6	5.65
			75000.	2000.	0.25	28.2	5.36
		100.	44000.	2000.	0.25	42.6	7.19
			52000.	2000.	0.25	39.2	6.74
			60000.	2000.	0.25	36.5	6.39
			75000.	2000.	0.25	32.6	6.05
		150.	44000.	2000.	0.25	52.1	8.60
			52000.	2000.	0.25	48.0	8.06
			60000.	2000.	0.25	44.7	7.63
			75000.	2000.	0.25	39.9	7.21
		200.	44000.	2000.	0.25	60.2	9.80
			52000.	2000.	0.25	55.4	9.17
			60000.	2000.	0.25	51.6	8.67
			75000.	2000.	0.25	46.1	8.19
		250.	44000.	2000.	0.25	67.3	10.85
			52000.	2000.	0.25	61.9	10.14
			60000.	2000.	0.25	57.7	9.59
			75000.	2000.	0.25	51.6	9.06
		300.	44000.	2000.	0.25	73.7	11.80
			52000.	2000.	0.25	67.8	11.03
			60000.	2000.	0.25	63.2	10.42
			75000.	2000.	0.25	56.5	9.84
		350.	44000.	2000.	0.25	79.7	12.68
			52000.	2000.	0.25	73.3	11.84
			60000.	2000.	0.25	68.2	11.19
			75000.	2000.	0.25	61.0	10.56

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
21.0	0.8	10.	44000.	2000.	0.25	14.1	2.96
			52000.	2000.	0.25	13.0	2.81
			60000.	2000.	0.25	12.1	2.70
			75000.	2000.	0.25	10.8	2.58
		25.	44000.	2000.	0.25	22.3	4.17
			52000.	2000.	0.25	20.5	3.94
			60000.	2000.	0.25	19.1	3.75
			75000.	2000.	0.25	17.1	3.57
		50.	44000.	2000.	0.25	31.5	5.54
			52000.	2000.	0.25	29.0	5.20
			60000.	2000.	0.25	27.0	4.94
			75000.	2000.	0.25	24.1	4.69
		75.	44000.	2000.	0.25	38.6	6.58
			52000.	2000.	0.25	35.5	6.17
			60000.	2000.	0.25	33.1	5.86
			75000.	2000.	0.25	29.6	5.55
		100.	44000.	2000.	0.25	44.6	7.46
			52000.	2000.	0.25	41.0	6.99
			60000.	2000.	0.25	38.2	6.63
			75000.	2000.	0.25	34.1	6.27
		150.	44000.	2000.	0.25	54.6	8.94
			52000.	2000.	0.25	50.2	8.37
			60000.	2000.	0.25	46.8	7.92
			75000.	2000.	0.25	41.8	7.48
		200.	44000.	2000.	0.25	63.1	10.19
			52000.	2000.	0.25	58.0	9.53
			60000.	2000.	0.25	54.0	9.01
			75000.	2000.	0.25	48.3	8.50
		250.	44000.	2000.	0.25	70.5	11.29
			52000.	2000.	0.25	64.8	10.55
			60000.	2000.	0.25	60.4	9.96
			75000.	2000.	0.25	54.0	9.40
		300.	44000.	2000.	0.25	77.2	12.28
			52000.	2000.	0.25	71.0	11.47
			60000.	2000.	0.25	66.1	10.83
			75000.	2000.	0.25	59.1	10.21
		350.	44000.	2000.	0.25	83.4	13.20
			52000.	2000.	0.25	76.7	12.32
			60000.	2000.	0.25	71.4	11.63
			75000.	2000.	0.25	63.9	10.96

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEDMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_s ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
21.0	0.7	10.	44000.	2000.	0.25	14.8	3.05
			52000.	2000.	0.25	13.6	2.90
			60000.	2000.	0.25	12.6	2.78
			75000.	2000.	0.25	11.3	2.66
		25.	44000.	2000.	0.25	23.3	4.32
			52000.	2000.	0.25	21.5	4.07
			60000.	2000.	0.25	20.0	3.88
			75000.	2000.	0.25	17.9	3.69
		50.	44000.	2000.	0.25	33.0	5.74
			52000.	2000.	0.25	30.4	5.39
			60000.	2000.	0.25	28.3	5.12
			75000.	2000.	0.25	25.3	4.85
		75.	44000.	2000.	0.25	40.4	6.83
			52000.	2000.	0.25	37.2	6.41
			60000.	2000.	0.25	34.6	6.07
			75000.	2000.	0.25	31.0	5.74
		100.	44000.	2000.	0.25	46.7	7.76
			52000.	2000.	0.25	43.0	7.26
			60000.	2000.	0.25	40.0	6.87
			75000.	2000.	0.25	35.8	6.49
		150.	44000.	2000.	0.25	57.2	9.30
			52000.	2000.	0.25	52.6	8.70
			60000.	2000.	0.25	49.0	8.22
			75000.	2000.	0.25	43.8	7.76
		200.	44000.	2000.	0.25	66.0	10.60
			52000.	2000.	0.25	60.7	9.90
			60000.	2000.	0.25	56.6	9.36
			75000.	2000.	0.25	50.6	8.82
		250.	44000.	2000.	0.25	73.8	11.75
			52000.	2000.	0.25	67.9	10.97
			60000.	2000.	0.25	63.2	10.36
			75000.	2000.	0.25	56.6	9.76
		300.	44000.	2000.	0.25	80.9	12.79
			52000.	2000.	0.25	74.4	11.93
			60000.	2000.	0.25	69.3	11.26
			75000.	2000.	0.25	61.9	10.61
		350.	44000.	2000.	0.25	87.4	13.74
			52000.	2000.	0.25	80.4	12.82
			60000.	2000.	0.25	74.8	12.09
			75000.	2000.	0.25	66.9	11.38

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED - END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(.0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_s ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
21.0	0.6	10.	44000.	2000.	0.25	15.5	3.15
			52000.	2000.	0.25	14.2	2.99
			60000.	2000.	0.25	13.2	2.86
			75000.	2000.	0.25	11.8	2.73
		25.	44000.	2000.	0.25	24.4	4.47
			52000.	2000.	0.25	22.5	4.21
			60000.	2000.	0.25	20.9	4.01
			75000.	2000.	0.25	18.7	3.81
		50.	44000.	2000.	0.25	34.6	5.96
			52000.	2000.	0.25	31.8	5.59
			60000.	2000.	0.25	29.6	5.30
			75000.	2000.	0.25	26.5	5.02
		75.	44000.	2000.	0.25	42.3	7.10
			52000.	2000.	0.25	39.0	6.65
			60000.	2000.	0.25	36.3	6.29
			75000.	2000.	0.25	32.4	5.95
		100.	44000.	2000.	0.25	48.9	8.06
			52000.	2000.	0.25	45.0	7.54
			60000.	2000.	0.25	41.9	7.13
			75000.	2000.	0.25	37.5	6.73
		150.	44000.	2000.	0.25	59.9	9.67
			52000.	2000.	0.25	55.1	9.04
			60000.	2000.	0.25	51.3	8.54
			75000.	2000.	0.25	45.9	8.04
		200.	44000.	2000.	0.25	69.2	11.03
			52000.	2000.	0.25	63.6	10.30
			60000.	2000.	0.25	59.2	9.72
			75000.	2000.	0.25	53.0	9.15
		250.	44000.	2000.	0.25	77.3	12.23
			52000.	2000.	0.25	71.1	11.41
			60000.	2000.	0.25	66.2	10.77
			75000.	2000.	0.25	59.2	10.13
		300.	44000.	2000.	0.25	84.7	13.31
			52000.	2000.	0.25	77.9	12.42
			60000.	2000.	0.25	72.5	11.71
			75000.	2000.	0.25	64.9	11.01
		350.	44000.	2000.	0.25	91.5	14.31
			52000.	2000.	0.25	84.1	13.34
			60000.	2000.	0.25	78.3	12.58
			75000.	2000.	0.25	70.1	11.82

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
24.5	1.0	10.	44000.	2000.	0.25	15.0	3.09
			52000.	2000.	0.25	13.8	2.93
			60000.	2000.	0.25	12.8	2.80
			75000.	2000.	0.25	11.5	2.68
		25.	44000.	2000.	0.25	23.7	4.37
			52000.	2000.	0.25	21.8	4.12
			60000.	2000.	0.25	20.3	3.92
			75000.	2000.	0.25	18.1	3.73
		50.	44000.	2000.	0.25	33.5	5.82
			52000.	2000.	0.25	30.8	5.46
			60000.	2000.	0.25	28.7	5.18
			75000.	2000.	0.25	25.7	4.91
		75.	44000.	2000.	0.25	41.0	6.93
			52000.	2000.	0.25	37.8	6.49
			60000.	2000.	0.25	35.1	6.15
			75000.	2000.	0.25	31.4	5.82
		100.	44000.	2000.	0.25	47.4	7.86
			52000.	2000.	0.25	43.6	7.36
			60000.	2000.	0.25	40.6	6.97
			75000.	2000.	0.25	36.3	6.58
		150.	44000.	2000.	0.25	58.0	9.43
			52000.	2000.	0.25	53.4	8.82
			60000.	2000.	0.25	49.7	8.33
			75000.	2000.	0.25	44.5	7.86
		200.	44000.	2000.	0.25	67.0	10.75
			52000.	2000.	0.25	61.6	10.04
			60000.	2000.	0.25	57.4	9.49
			75000.	2000.	0.25	51.3	8.94
		250.	44000.	2000.	0.25	74.9	11.92
			52000.	2000.	0.25	68.9	11.12
			60000.	2000.	0.25	64.2	10.50
			75000.	2000.	0.25	57.4	9.90
		300.	44000.	2000.	0.25	82.1	12.97
			52000.	2000.	0.25	75.5	12.10
			60000.	2000.	0.25	70.3	11.42
			75000.	2000.	0.25	62.9	10.76
		350.	44000.	2000.	0.25	88.7	13.94
			52000.	2000.	0.25	81.6	13.00
			60000.	2000.	0.25	75.9	12.27
			75000.	2000.	0.25	67.9	11.55

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
24.5	0.9	10.	44000.	2000.	0.25	15.7	3.19
			52000.	2000.	0.25	14.5	3.02
			60000.	2000.	0.25	13.5	2.89
			75000.	2000.	0.25	12.0	2.76
		25.	44000.	2000.	0.25	24.8	4.53
			52000.	2000.	0.25	22.8	4.27
			60000.	2000.	0.25	21.3	4.06
			75000.	2000.	0.25	19.0	3.86
		50.	44000.	2000.	0.25	35.1	6.04
			52000.	2000.	0.25	32.3	5.67
			60000.	2000.	0.25	30.1	5.38
			75000.	2000.	0.25	26.9	5.09
		75.	44000.	2000.	0.25	43.0	7.20
			52000.	2000.	0.25	39.6	6.75
			60000.	2000.	0.25	36.8	6.39
			75000.	2000.	0.25	33.0	6.03
		100.	44000.	2000.	0.25	49.7	8.18
			52000.	2000.	0.25	45.7	7.65
			60000.	2000.	0.25	42.5	7.24
			75000.	2000.	0.25	38.0	6.83
		150.	44000.	2000.	0.25	60.8	9.82
			52000.	2000.	0.25	56.0	9.17
			60000.	2000.	0.25	52.1	8.67
			75000.	2000.	0.25	46.6	8.17
		200.	44000.	2000.	0.25	70.3	11.20
			52000.	2000.	0.25	64.6	10.46
			60000.	2000.	0.25	60.2	9.87
			75000.	2000.	0.25	53.8	9.30
		250.	44000.	2000.	0.25	78.5	12.42
			52000.	2000.	0.25	72.3	11.59
			60000.	2000.	0.25	67.3	10.93
			75000.	2000.	0.25	60.2	10.29
		300.	44000.	2000.	0.25	86.0	13.52
			52000.	2000.	0.25	79.1	12.61
			60000.	2000.	0.25	73.7	11.89
			75000.	2000.	0.25	65.9	11.19
		350.	44000.	2000.	0.25	92.9	14.54
			52000.	2000.	0.25	85.5	13.55
			60000.	2000.	0.25	79.6	12.78
			75000.	2000.	0.25	71.2	12.01

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(.0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
24.5	0.8	10.	44000.	2000.	0.25	16.4	3.29
			52000.	2000.	0.25	15.1	3.12
			60000.	2000.	0.25	14.1	2.98
			75000.	2000.	0.25	12.6	2.84
		25.	44000.	2000.	0.25	26.0	4.69
			52000.	2000.	0.25	23.9	4.42
			60000.	2000.	0.25	22.3	4.20
			75000.	2000.	0.25	19.9	3.98
		50.	44000.	2000.	0.25	36.8	6.27
			52000.	2000.	0.25	33.8	5.88
			60000.	2000.	0.25	31.5	5.57
			75000.	2000.	0.25	28.2	5.27
		75.	44000.	2000.	0.25	45.0	7.48
			52000.	2000.	0.25	41.4	7.00
			60000.	2000.	0.25	38.6	6.63
			75000.	2000.	0.25	34.5	6.25
		100.	44000.	2000.	0.25	52.0	8.50
			52000.	2000.	0.25	47.8	7.95
			60000.	2000.	0.25	44.5	7.52
			75000.	2000.	0.25	39.8	7.08
		150.	44000.	2000.	0.25	63.7	10.22
			52000.	2000.	0.25	58.6	9.54
			60000.	2000.	0.25	54.6	9.01
			75000.	2000.	0.25	48.8	8.48
		200.	44000.	2000.	0.25	73.6	11.66
			52000.	2000.	0.25	67.7	10.88
			60000.	2000.	0.25	63.0	10.26
			75000.	2000.	0.25	56.3	9.65
		250.	44000.	2000.	0.25	82.2	12.93
			52000.	2000.	0.25	75.7	12.06
			60000.	2000.	0.25	70.4	11.37
			75000.	2000.	0.25	63.0	10.69
		300.	44000.	2000.	0.25	90.1	14.08
			52000.	2000.	0.25	82.9	13.13
			60000.	2000.	0.25	77.2	12.37
			75000.	2000.	0.25	69.0	11.63
		350.	44000.	2000.	0.25	97.3	15.14
			52000.	2000.	0.25	89.5	14.11
			60000.	2000.	0.25	83.3	13.29
			75000.	2000.	0.25	74.5	12.49

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
24.5	0.7	10.	44000.	2000.	0.25	17.2	3.40
			52000.	2000.	0.25	15.8	3.21
			60000.	2000.	0.25	14.8	3.07
			75000.	2000.	0.25	13.2	2.93
		25.	44000.	2000.	0.25	27.2	4.86
			52000.	2000.	0.25	25.1	4.57
			60000.	2000.	0.25	23.3	4.34
			75000.	2000.	0.25	20.9	4.11
		50.	44000.	2000.	0.25	38.5	6.51
			52000.	2000.	0.25	35.4	6.10
			60000.	2000.	0.25	33.0	5.78
			75000.	2000.	0.25	29.5	5.45
		75.	44000.	2000.	0.25	47.2	7.78
			52000.	2000.	0.25	43.4	7.27
			60000.	2000.	0.25	40.4	6.88
			75000.	2000.	0.25	36.1	6.48
		100.	44000.	2000.	0.25	54.5	8.84
			52000.	2000.	0.25	50.1	8.26
			60000.	2000.	0.25	46.7	7.81
			75000.	2000.	0.25	41.7	7.35
		150.	44000.	2000.	0.25	66.7	10.63
			52000.	2000.	0.25	61.4	9.92
			60000.	2000.	0.25	57.1	9.36
			75000.	2000.	0.25	51.1	8.80
		200.	44000.	2000.	0.25	77.0	12.14
			52000.	2000.	0.25	70.9	11.32
			60000.	2000.	0.25	66.0	10.68
			75000.	2000.	0.25	59.0	10.03
		250.	44000.	2000.	0.25	86.1	13.47
			52000.	2000.	0.25	79.2	12.55
			60000.	2000.	0.25	73.8	11.83
			75000.	2000.	0.25	66.0	11.11
		300.	44000.	2000.	0.25	94.4	14.67
			52000.	2000.	0.25	86.8	13.67
			60000.	2000.	0.25	80.8	12.88
			75000.	2000.	0.25	72.3	12.09
		350.	44000.	2000.	0.25	101.9	15.78
			52000.	2000.	0.25	93.8	14.69
			60000.	2000.	0.25	87.3	13.84
			75000.	2000.	0.25	78.1	12.98

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(.0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} %	D in.	Ct $/sq ft
24.5	0.6	10.	44000.	2000.	0.25	18.0	3.51
			52000.	2000.	0.25	16.6	3.32
			60000.	2000.	0.25	15.4	3.17
			75000.	2000.	0.25	13.8	3.01
		25.	44000.	2000.	0.25	28.5	5.04
			52000.	2000.	0.25	26.2	4.73
			60000.	2000.	0.25	24.4	4.49
			75000.	2000.	0.25	21.8	4.25
		50.	44000.	2000.	0.25	40.3	6.76
			52000.	2000.	0.25	37.1	6.33
			60000.	2000.	0.25	34.5	5.99
			75000.	2000.	0.25	30.9	5.65
		75.	44000.	2000.	0.25	49.4	8.08
			52000.	2000.	0.25	45.4	7.56
			60000.	2000.	0.25	42.3	7.14
			75000.	2000.	0.25	37.8	6.72
		100.	44000.	2000.	0.25	57.0	9.20
			52000.	2000.	0.25	52.5	8.59
			60000.	2000.	0.25	48.9	8.11
			75000.	2000.	0.25	43.7	7.63
		150.	44000.	2000.	0.25	69.9	11.07
			52000.	2000.	0.25	64.3	10.32
			60000.	2000.	0.25	59.8	9.73
			75000.	2000.	0.25	53.5	9.14
		200.	44000.	2000.	0.25	80.7	12.64
			52000.	2000.	0.25	74.2	11.78
			60000.	2000.	0.25	69.1	11.10
			75000.	2000.	0.25	61.8	10.42
		250.	44000.	2000.	0.25	90.2	14.03
			52000.	2000.	0.25	83.0	13.07
			60000.	2000.	0.25	77.2	12.31
			75000.	2000.	0.25	69.1	11.55
		300.	44000.	2000.	0.25	98.8	15.29
			52000.	2000.	0.25	90.9	14.23
			60000.	2000.	0.25	84.6	13.40
			75000.	2000.	0.25	75.7	12.56
		350.	44000.	2000.	0.25	106.7	16.44
			52000.	2000.	0.25	98.2	15.30
			60000.	2000.	0.25	91.4	14.40
			75000.	2000.	0.25	81.7	13.50

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
28.0	1.0	10.	44000.	2000.	0.25	17.1	3.39
			52000.	2000.	0.25	15.8	3.20
			60000.	2000.	0.25	14.7	3.06
			75000.	2000.	0.25	13.1	2.92
		25.	44000.	2000.	0.25	27.1	4.84
			52000.	2000.	0.25	24.9	4.56
			60000.	2000.	0.25	23.2	4.33
			75000.	2000.	0.25	20.7	4.10
		50.	44000.	2000.	0.25	38.3	6.49
			52000.	2000.	0.25	35.2	6.08
			60000.	2000.	0.25	32.8	5.76
			75000.	2000.	0.25	29.3	5.44
		75.	44000.	2000.	0.25	46.9	7.75
			52000.	2000.	0.25	43.1	7.25
			60000.	2000.	0.25	40.2	6.85
			75000.	2000.	0.25	35.9	6.46
		100.	44000.	2000.	0.25	54.2	8.81
			52000.	2000.	0.25	49.8	8.23
			60000.	2000.	0.25	46.4	7.78
			75000.	2000.	0.25	41.5	7.33
		150.	44000.	2000.	0.25	66.3	10.59
			52000.	2000.	0.25	61.0	9.88
			60000.	2000.	0.25	56.8	9.33
			75000.	2000.	0.25	50.8	8.77
		200.	44000.	2000.	0.25	76.6	12.09
			52000.	2000.	0.25	70.5	11.28
			60000.	2000.	0.25	65.6	10.63
			75000.	2000.	0.25	58.7	10.00
		250.	44000.	2000.	0.25	85.6	13.41
			52000.	2000.	0.25	78.8	12.50
			60000.	2000.	0.25	73.3	11.79
			75000.	2000.	0.25	65.6	11.07
		300.	44000.	2000.	0.25	93.8	14.61
			52000.	2000.	0.25	86.3	13.61
			60000.	2000.	0.25	80.3	12.83
			75000.	2000.	0.25	71.8	12.05
		350.	44000.	2000.	0.25	101.3	15.71
			52000.	2000.	0.25	93.2	14.63
			60000.	2000.	0.25	86.8	13.78
			75000.	2000.	0.25	77.6	12.94

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000, \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
28.0	0.9	10.	44000.	2000.	0.25	18.0	3.50
			52000.	2000.	0.25	16.5	3.31
			60000.	2000.	0.25	15.4	3.16
			75000.	2000.	0.25	13.8	3.01
		25.	44000.	2000.	0.25	28.4	5.03
			52000.	2000.	0.25	26.1	4.72
			60000.	2000.	0.25	24.3	4.48
			75000.	2000.	0.25	21.7	4.25
		50.	44000.	2000.	0.25	40.1	6.74
			52000.	2000.	0.25	36.9	6.31
			60000.	2000.	0.25	34.4	5.98
			75000.	2000.	0.25	30.7	5.64
		75.	44000.	2000.	0.25	49.2	8.06
			52000.	2000.	0.25	45.2	7.54
			60000.	2000.	0.25	42.1	7.12
			75000.	2000.	0.25	37.7	6.71
		100.	44000.	2000.	0.25	56.8	9.17
			52000.	2000.	0.25	52.2	8.57
			60000.	2000.	0.25	48.6	8.09
			75000.	2000.	0.25	43.5	7.61
		150.	44000.	2000.	0.25	69.5	11.04
			52000.	2000.	0.25	64.0	10.29
			60000.	2000.	0.25	59.5	9.71
			75000.	2000.	0.25	53.3	9.12
		200.	44000.	2000.	0.25	80.3	12.61
			52000.	2000.	0.25	73.9	11.75
			60000.	2000.	0.25	68.8	11.08
			75000.	2000.	0.25	61.5	10.40
		250.	44000.	2000.	0.25	89.8	13.99
			52000.	2000.	0.25	82.6	13.03
			60000.	2000.	0.25	76.9	12.28
			75000.	2000.	0.25	68.8	11.52
		300.	44000.	2000.	0.25	98.3	15.24
			52000.	2000.	0.25	90.5	14.19
			60000.	2000.	0.25	84.2	13.37
			75000.	2000.	0.25	75.3	12.54
		350.	44000.	2000.	0.25	106.2	16.39
			52000.	2000.	0.25	97.7	15.26
			60000.	2000.	0.25	91.0	14.37
			75000.	2000.	0.25	81.4	13.47

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
28.0	0.8	10.	44000.	2000.	0.25	18.8	3.62
			52000.	2000.	0.25	17.3	3.42
			60000.	2000.	0.25	16.1	3.26
			75000.	2000.	0.25	14.4	3.10
		25.	44000.	2000.	0.25	29.7	5.21
			52000.	2000.	0.25	27.3	4.89
			60000.	2000.	0.25	25.5	4.64
			75000.	2000.	0.25	22.8	4.39
		50.	44000.	2000.	0.25	42.0	7.01
			52000.	2000.	0.25	38.7	6.56
			60000.	2000.	0.25	36.0	6.20
			75000.	2000.	0.25	32.2	5.84
		75.	44000.	2000.	0.25	51.5	8.38
			52000.	2000.	0.25	47.4	7.83
			60000.	2000.	0.25	44.1	7.40
			75000.	2000.	0.25	39.4	6.96
		100.	44000.	2000.	0.25	59.4	9.54
			52000.	2000.	0.25	54.7	8.91
			60000.	2000.	0.25	50.9	8.41
			75000.	2000.	0.25	45.5	7.90
		150.	44000.	2000.	0.25	72.8	11.49
			52000.	2000.	0.25	67.0	10.71
			60000.	2000.	0.25	62.3	10.10
			75000.	2000.	0.25	55.8	9.48
		200.	44000.	2000.	0.25	84.1	13.13
			52000.	2000.	0.25	77.3	12.23
			60000.	2000.	0.25	72.0	11.52
			75000.	2000.	0.25	64.4	10.81
		250.	44000.	2000.	0.25	94.0	14.58
			52000.	2000.	0.25	86.5	13.57
			60000.	2000.	0.25	80.5	12.78
			75000.	2000.	0.25	72.0	11.98
		300.	44000.	2000.	0.25	103.0	15.89
			52000.	2000.	0.25	94.7	14.78
			60000.	2000.	0.25	88.2	13.92
			75000.	2000.	0.25	78.9	13.04
		350.	44000.	2000.	0.25	111.2	17.09
			52000.	2000.	0.25	102.3	15.90
			60000.	2000.	0.25	95.2	14.96
			75000.	2000.	0.25	85.2	14.02

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_s ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
28.0	0.7	10.	44000.	2000.	0.25	19.7	3.74
			52000.	2000.	0.25	18.1	3.53
			60000.	2000.	0.25	16.9	3.37
			75000.	2000.	0.25	15.1	3.20
		25.	44000.	2000.	0.25	31.1	5.41
			52000.	2000.	0.25	28.6	5.07
			60000.	2000.	0.25	26.7	4.81
			75000.	2000.	0.25	23.8	4.54
		50.	44000.	2000.	0.25	44.0	7.28
			52000.	2000.	0.25	40.5	6.81
			60000.	2000.	0.25	37.7	6.44
			75000.	2000.	0.25	33.7	6.06
		75.	44000.	2000.	0.25	53.9	8.72
			52000.	2000.	0.25	49.6	8.14
			60000.	2000.	0.25	46.2	7.69
			75000.	2000.	0.25	41.3	7.22
		100.	44000.	2000.	0.25	62.3	9.93
			52000.	2000.	0.25	57.3	9.26
			60000.	2000.	0.25	53.3	8.74
			75000.	2000.	0.25	47.7	8.20
		150.	44000.	2000.	0.25	76.3	11.97
			52000.	2000.	0.25	70.1	11.15
			60000.	2000.	0.25	65.3	10.51
			75000.	2000.	0.25	58.4	9.85
		200.	44000.	2000.	0.25	88.0	13.68
			52000.	2000.	0.25	81.0	12.74
			60000.	2000.	0.25	75.4	11.99
			75000.	2000.	0.25	67.4	11.24
		250.	44000.	2000.	0.25	98.4	15.19
			52000.	2000.	0.25	90.6	14.14
			60000.	2000.	0.25	84.3	13.31
			75000.	2000.	0.25	75.4	12.46
		300.	44000.	2000.	0.25	107.8	16.56
			52000.	2000.	0.25	99.2	15.40
			60000.	2000.	0.25	92.3	14.49
			75000.	2000.	0.25	82.6	13.57
		350.	44000.	2000.	0.25	116.5	17.82
			52000.	2000.	0.25	107.1	16.57
			60000.	2000.	0.25	99.7	15.58
			75000.	2000.	0.25	89.2	14.58

TABLE 3-39 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = \phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
28.0	0.6	10.	44000.	2000.	0.25	20.6	3.87
			52000.	2000.	0.25	19.0	3.65
			60000.	2000.	0.25	17.7	3.47
			75000.	2000.	0.25	15.8	3.30
		25.	44000.	2000.	0.25	32.6	5.61
			52000.	2000.	0.25	30.0	5.26
			60000.	2000.	0.25	27.9	4.98
			75000.	2000.	0.25	25.0	4.70
		50.	44000.	2000.	0.25	46.1	7.57
			52000.	2000.	0.25	42.4	7.07
			60000.	2000.	0.25	39.5	6.68
			75000.	2000.	0.25	35.3	6.28
		75.	44000.	2000.	0.25	56.5	9.07
			52000.	2000.	0.25	51.9	8.46
			60000.	2000.	0.25	48.4	7.99
			75000.	2000.	0.25	43.2	7.50
		100.	44000.	2000.	0.25	65.2	10.34
			52000.	2000.	0.25	60.0	9.64
			60000.	2000.	0.25	55.8	9.09
			75000.	2000.	0.25	49.9	8.52
		150.	44000.	2000.	0.25	79.9	12.47
			52000.	2000.	0.25	73.5	11.61
			60000.	2000.	0.25	68.4	10.93
			75000.	2000.	0.25	61.2	10.24
		200.	44000.	2000.	0.25	92.2	14.26
			52000.	2000.	0.25	84.8	13.27
			60000.	2000.	0.25	79.0	12.48
			75000.	2000.	0.25	70.6	11.69
		250.	44000.	2000.	0.25	103.1	15.84
			52000.	2000.	0.25	94.8	14.73
			60000.	2000.	0.25	88.3	13.85
			75000.	2000.	0.25	79.0	12.96
		300.	44000.	2000.	0.25	112.9	17.26
			52000.	2000.	0.25	103.9	16.05
			60000.	2000.	0.25	96.7	15.09
			75000.	2000.	0.25	86.5	14.11
		350.	44000.	2000.	0.25	122.0	18.58
			52000.	2000.	0.25	112.2	17.26
			60000.	2000.	0.25	104.5	16.23
			75000.	2000.	0.25	93.4	15.17

TABLE 3-40

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\ D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
7.0	1.0	10.	44000.	3000.	0.25	4.5	1.62
			52000.	3000.	0.25	4.5	1.63
			60000.	3500.	0.25	4.5	1.64
			75000.	4000.	0.25	4.5	1.69
		25.	44000.	6000.	0.38	5.5	1.97
			52000.	3000.	0.25	6.2	1.92
			60000.	3500.	0.25	5.8	1.86
			75000.	4000.	0.25	5.2	1.81
		50.	44000.	6000.	0.38	7.8	2.43
			52000.	3000.	0.25	8.8	2.35
			60000.	3500.	0.25	8.2	2.27
			75000.	4000.	0.25	7.3	2.20
		75.	44000.	6000.	0.38	9.6	2.78
			52000.	3000.	0.25	10.8	2.68
			60000.	3500.	0.25	10.0	2.58
			75000.	4000.	0.25	9.0	2.50
		100.	44000.	6000.	0.38	11.0	3.07
			52000.	3000.	0.25	12.5	2.96
			60000.	3500.	0.25	11.6	2.85
			75000.	4000.	0.25	10.4	2.75
		150.	44000.	6000.	0.38	13.5	3.56
			52000.	3000.	0.25	15.3	3.42
			60000.	3500.	0.25	14.2	3.29
			75000.	4000.	0.25	12.7	3.17
		200.	44000.	6000.	0.38	15.6	3.98
			52000.	3000.	0.25	17.6	3.82
			60000.	3500.	0.25	16.4	3.66
			75000.	4000.	0.25	14.7	3.52
		250.	44000.	6000.	0.38	17.5	4.34
			52000.	3000.	0.25	19.7	4.16
			60000.	3500.	0.25	18.3	3.99
			75000.	4000.	0.25	16.4	3.83
		300.	44000.	6000.	0.38	19.1	4.67
			52000.	3000.	0.25	21.6	4.48
			60000.	3500.	0.25	20.1	4.29
			75000.	4000.	0.25	18.0	4.12
		350.	44000.	6000.	0.38	20.7	4.98
			52000.	3000.	0.25	23.3	4.76
			60000.	3500.	0.25	21.7	4.56
			75000.	4000.	0.25	19.4	4.38

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9$ D)

L_s ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
7.0	0.9	10.	44000.	2500.	0.25	4.5	1.56
			52000.	2500.	0.25	4.5	1.57
			60000.	3000.	0.25	4.5	1.58
			75000.	3500.	0.25	4.5	1.62
		25.	44000.	6000.	0.41	5.5	1.92
			52000.	2500.	0.25	6.5	1.88
			60000.	3000.	0.25	6.1	1.83
			75000.	3500.	0.25	5.4	1.78
		50.	44000.	6000.	0.41	7.8	2.35
			52000.	2500.	0.25	9.2	2.30
			60000.	3000.	0.25	8.6	2.22
			75000.	3500.	0.25	7.7	2.15
		75.	44000.	6000.	0.41	9.6	2.69
			52000.	2500.	0.25	11.3	2.62
			60000.	3000.	0.25	10.5	2.52
			75000.	3500.	0.25	9.4	2.43
		100.	44000.	6000.	0.41	11.1	2.97
			52000.	2500.	0.25	13.1	2.89
			60000.	3000.	0.25	12.2	2.78
			75000.	3500.	0.25	10.9	2.68
		150.	44000.	6000.	0.41	13.6	3.43
			52000.	2500.	0.25	16.0	3.34
			60000.	3000.	0.25	14.9	3.20
			75000.	3500.	0.25	13.3	3.08
		200.	44000.	6000.	0.41	15.7	3.83
			52000.	2500.	0.25	18.5	3.72
			60000.	3000.	0.25	17.2	3.56
			75000.	3500.	0.25	15.4	3.42
		250.	44000.	6000.	0.41	17.5	4.18
			52000.	2500.	0.25	20.6	4.05
			60000.	3000.	0.25	19.2	3.88
			75000.	3500.	0.25	17.2	3.72
		300.	44000.	6000.	0.41	19.2	4.49
			52000.	2500.	0.25	22.6	4.36
			60000.	3000.	0.25	21.1	4.17
			75000.	3500.	0.25	18.8	3.99
		350.	44000.	6000.	0.41	20.7	4.78
			52000.	2500.	0.25	24.4	4.63
			60000.	3000.	0.25	22.7	4.43
			75000.	3500.	0.25	20.3	4.24

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\,D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
7.0	0.8	10.	44000.	2500.	0.25	4.7	1.56
			52000.	2500.	0.25	4.5	1.54
			60000.	2500.	0.25	4.5	1.55
			75000.	3000.	0.25	4.5	1.59
		25.	44000.	6000.	0.44	5.6	1.91
			52000.	2500.	0.25	6.8	1.88
			60000.	2500.	0.25	6.4	1.83
			75000.	3000.	0.25	5.7	1.77
		50.	44000.	6000.	0.44	7.9	2.33
			52000.	2500.	0.25	9.7	2.30
			60000.	2500.	0.25	9.0	2.22
			75000.	3000.	0.25	8.0	2.14
		75.	44000.	6000.	0.44	9.7	2.66
			52000.	2500.	0.25	11.8	2.62
			60000.	2500.	0.25	11.0	2.52
			75000.	3000.	0.25	9.9	2.43
		100.	44000.	6000.	0.44	11.2	2.93
			52000.	2500.	0.25	13.7	2.88
			60000.	2500.	0.25	12.7	2.78
			75000.	3000.	0.25	11.4	2.67
		150.	44000.	6000.	0.44	13.7	3.40
			52000.	2500.	0.25	16.7	3.33
			60000.	2500.	0.25	15.6	3.20
			75000.	3000.	0.25	13.9	3.07
		200.	44000.	6000.	0.44	15.8	3.78
			52000.	2500.	0.25	19.3	3.71
			60000.	2500.	0.25	18.0	3.56
			75000.	3000.	0.25	16.1	3.41
		250.	44000.	6000.	0.44	17.7	4.13
			52000.	2500.	0.25	21.6	4.05
			60000.	2500.	0.25	20.1	3.88
			75000.	3000.	0.25	18.0	3.71
		300.	44000.	6000.	0.44	19.4	4.44
			52000.	2500.	0.25	23.7	4.35
			60000.	2500.	0.25	22.0	4.17
			75000.	3000.	0.25	19.7	3.98
		350.	44000.	6000.	0.44	20.9	4.72
			52000.	2500.	0.25	25.6	4.63
			60000.	2500.	0.25	23.8	4.43
			75000.	3000.	0.25	21.3	4.23

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\,D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
7.0	0.7	10.	44000.	3500.	0.28	4.7	1.57
			52000.	2500.	0.25	4.5	1.53
			60000.	2500.	0.25	4.5	1.53
			75000.	3000.	0.25	4.5	1.57
		25.	44000.	6000.	0.47	5.7	1.91
			52000.	2500.	0.25	7.2	1.90
			60000.	2500.	0.25	6.7	1.84
			75000.	3000.	0.25	6.0	1.79
		50.	44000.	6000.	0.47	8.0	2.33
			52000.	2500.	0.25	10.1	2.32
			60000.	2500.	0.25	9.4	2.24
			75000.	3000.	0.25	8.4	2.16
		75.	44000.	6000.	0.47	9.8	2.66
			52000.	2500.	0.25	12.4	2.65
			60000.	2500.	0.25	11.5	2.55
			75000.	3000.	0.25	10.3	2.45
		100.	44000.	6000.	0.47	11.3	2.94
			52000.	2500.	0.25	14.3	2.92
			60000.	2500.	0.25	13.3	2.81
			75000.	3000.	0.25	11.9	2.70
		150.	44000.	6000.	0.47	13.8	3.40
			52000.	2500.	0.25	17.5	3.38
			60000.	2500.	0.25	16.3	3.24
			75000.	3000.	0.25	14.6	3.10
		200.	44000.	6000.	0.47	16.0	3.79
			52000.	2500.	0.25	20.2	3.77
			60000.	2500.	0.25	18.9	3.61
			75000.	3000.	0.25	16.9	3.45
		250.	44000.	6000.	0.47	17.9	4.13
			52000.	2500.	0.25	22.6	4.11
			60000.	2500.	0.25	21.1	3.93
			75000.	3000.	0.25	18.9	3.75
		300.	44000.	6000.	0.47	19.6	4.44
			52000.	2500.	0.25	24.8	4.41
			60000.	2500.	0.25	23.1	4.22
			75000.	3000.	0.25	20.6	4.02
		350.	44000.	6000.	0.47	21.1	4.73
			52000.	2500.	0.25	26.8	4.70
			60000.	2500.	0.25	24.9	4.49
			75000.	3000.	0.25	22.3	4.28

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9$ D)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
7.0	0.6	10.	44000.	2500.	0.35	4.5	1.56
			52000.	2500.	0.35	4.5	1.58
			60000.	3000.	0.35	4.5	1.59
			75000.	3500.	0.35	4.5	1.63
		25.	44000.	6000.	0.50	5.7	1.92
			52000.	2500.	0.35	6.3	1.86
			60000.	3000.	0.35	5.9	1.80
			75000.	3500.	0.35	5.3	1.75
		50.	44000.	6000.	0.50	8.1	2.35
			52000.	2500.	0.35	8.9	2.26
			60000.	3000.	0.35	8.3	2.19
			75000.	3500.	0.35	7.4	2.12
		75.	44000.	6000.	0.50	9.9	2.68
			52000.	2500.	0.35	10.9	2.57
			60000.	3000.	0.35	10.2	2.48
			75000.	3500.	0.35	9.1	2.39
		100.	44000.	6000.	0.50	11.5	2.96
			52000.	2500.	0.35	12.6	2.83
			60000.	3000.	0.35	11.8	2.73
			75000.	3500.	0.35	10.5	2.63
		150.	44000.	6000.	0.50	14.0	3.43
			52000.	2500.	0.35	15.5	3.27
			60000.	3000.	0.35	14.4	3.14
			75000.	3500.	0.35	12.9	3.02
		200.	44000.	6000.	0.50	16.2	3.83
			52000.	2500.	0.35	17.9	3.64
			60000.	3000.	0.35	16.6	3.49
			75000.	3500.	0.35	14.9	3.35
		250.	44000.	6000.	0.50	18.1	4.17
			52000.	2500.	0.35	20.0	3.97
			60000.	3000.	0.35	18.6	3.80
			75000.	3500.	0.35	16.6	3.64
		300.	44000.	6000.	0.50	19.9	4.49
			52000.	2500.	0.35	21.9	4.26
			60000.	3000.	0.35	20.4	4.08
			75000.	3500.	0.35	18.2	3.91
		350.	44000.	6000.	0.50	21.5	4.78
			52000.	2500.	0.35	23.6	4.54
			60000.	3000.	0.35	22.0	4.34
			75000.	3500.	0.35	19.7	4.15

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
10.5	1.0	10.	44000.	2500.	0.25	6.4	1.89
			52000.	2500.	0.25	5.9	1.82
			60000.	2500.	0.25	5.5	1.77
			75000.	3000.	0.25	4.9	1.72
		25.	44000.	2500.	0.25	10.2	2.47
			52000.	2500.	0.25	9.3	2.37
			60000.	2500.	0.25	8.7	2.29
			75000.	3000.	0.25	7.8	2.21
		50.	44000.	2500.	0.25	14.4	3.13
			52000.	2500.	0.25	13.2	2.99
			60000.	2500.	0.25	12.3	2.87
			75000.	3000.	0.25	11.0	2.76
		75.	44000.	2500.	0.25	17.6	3.64
			52000.	2500.	0.25	16.2	3.46
			60000.	2500.	0.25	15.1	3.32
			75000.	3000.	0.25	13.5	3.19
		100.	44000.	2500.	0.25	20.3	4.07
			52000.	2500.	0.25	18.7	3.86
			60000.	2500.	0.25	17.4	3.70
			75000.	3000.	0.25	15.6	3.54
		150.	44000.	2500.	0.25	24.9	4.78
			52000.	2500.	0.25	22.9	4.53
			60000.	2500.	0.25	21.3	4.33
			75000.	3000.	0.25	19.1	4.14
		200.	44000.	2500.	0.25	28.7	5.39
			52000.	2500.	0.25	26.4	5.09
			60000.	2500.	0.25	24.6	4.86
			75000.	3000.	0.25	22.0	4.64
		250.	44000.	2500.	0.25	32.1	5.92
			52000.	2500.	0.25	29.5	5.59
			60000.	2500.	0.25	27.5	5.33
			75000.	3000.	0.25	24.6	5.09
		300.	44000.	2500.	0.25	35.2	6.40
			52000.	2500.	0.25	32.4	6.04
			60000.	2500.	0.25	30.1	5.76
			75000.	3000.	0.25	26.9	5.49
		350.	44000.	2500.	0.25	38.0	6.84
			52000.	2500.	0.25	35.0	6.45
			60000.	2500.	0.25	32.5	6.15
			75000.	3000.	0.25	29.1	5.86

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\,D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
10.5	0.9	10.	44000.	6000.	0.41	5.3	1.85
			52000.	2000.	0.25	6.2	1.79
			60000.	2500.	0.25	5.8	1.74
			75000.	3000.	0.25	5.2	1.69
		25.	44000.	6000.	0.41	8.3	2.41
			52000.	2000.	0.25	9.8	2.32
			60000.	2500.	0.25	9.1	2.24
			75000.	3000.	0.25	8.2	2.16
		50.	44000.	6000.	0.41	11.8	3.05
			52000.	2000.	0.25	13.8	2.92
			60000.	2500.	0.25	12.9	2.81
			75000.	3000.	0.25	11.5	2.69
		75.	44000.	6000.	0.41	14.4	3.54
			52000.	2000.	0.25	17.0	3.38
			60000.	2500.	0.25	15.8	3.24
			75000.	3000.	0.25	14.1	3.10
		100.	44000.	6000.	0.41	16.6	3.95
			52000.	2000.	0.25	19.6	3.77
			60000.	2500.	0.25	18.2	3.60
			75000.	3000.	0.25	16.3	3.44
		150.	44000.	6000.	0.41	20.4	4.64
			52000.	2000.	0.25	24.0	4.42
			60000.	2500.	0.25	22.3	4.22
			75000.	3000.	0.25	20.0	4.02
		200.	44000.	6000.	0.41	23.5	5.22
			52000.	2000.	0.25	27.7	4.96
			60000.	2500.	0.25	25.8	4.73
			75000.	3000.	0.25	23.1	4.51
		250.	44000.	6000.	0.41	26.3	5.73
			52000.	2000.	0.25	31.0	5.44
			60000.	2500.	0.25	28.8	5.19
			75000.	3000.	0.25	25.8	4.94
		300.	44000.	6000.	0.41	28.8	6.20
			52000.	2000.	0.25	33.9	5.88
			60000.	2500.	0.25	31.6	5.60
			75000.	3000.	0.25	28.2	5.32
		350.	44000.	6000.	0.41	31.1	6.62
			52000.	2000.	0.25	36.6	6.28
			60000.	2500.	0.25	34.1	5.98
			75000.	3000.	0.25	30.5	5.68

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\, \phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\,D$)

L_S ft	σ	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
10.5	0.8	10.	44000.	6000.	0.44	5.3	1.84
			52000.	2000.	0.25	6.5	1.79
			60000.	2000.	0.25	6.0	1.74
			75000.	2500.	0.25	5.4	1.69
		25.	44000.	6000.	0.44	8.4	2.39
			52000.	2000.	0.25	10.3	2.32
			60000.	2000.	0.25	9.5	2.24
			75000.	2500.	0.25	8.5	2.16
		50.	44000.	6000.	0.44	11.9	3.02
			52000.	2000.	0.25	14.5	2.92
			60000.	2000.	0.25	13.5	2.81
			75000.	2500.	0.25	12.1	2.69
		75.	44000.	6000.	0.44	14.5	3.50
			52000.	2000.	0.25	17.8	3.38
			60000.	2000.	0.25	16.5	3.24
			75000.	2500.	0.25	14.8	3.09
		100.	44000.	6000.	0.44	16.8	3.91
			52000.	2000.	0.25	20.5	3.77
			60000.	2000.	0.25	19.1	3.61
			75000.	2500.	0.25	17.1	3.44
		150.	44000.	6000.	0.44	20.5	4.59
			52000.	2000.	0.25	25.1	4.42
			60000.	2000.	0.25	23.4	4.22
			75000.	2500.	0.25	20.9	4.01
		200.	44000.	6000.	0.44	23.7	5.16
			52000.	2000.	0.25	29.0	4.96
			60000.	2000.	0.25	27.0	4.74
			75000.	2500.	0.25	24.1	4.50
		250.	44000.	6000.	0.44	26.5	5.66
			52000.	2000.	0.25	32.4	5.44
			60000.	2000.	0.25	30.2	5.19
			75000.	2500.	0.25	27.0	4.92
		300.	44000.	6000.	0.44	29.0	6.12
			52000.	2000.	0.25	35.5	5.88
			60000.	2000.	0.25	33.1	5.60
			75000.	2500.	0.25	29.6	5.31
		350.	44000.	6000.	0.44	31.4	6.54
			52000.	2000.	0.25	38.4	6.28
			60000.	2000.	0.25	35.7	5.98
			75000.	2500.	0.25	31.9	5.66

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

$\bar{L}_S$ ft	σ	q psi	f_{dy} psi	$\bar{f'}_c$ psi	$\bar{\phi}_{Sc}$ psi	D in.	Ct $/sq ft
10.5	0.7	10.	44000.	6000.	0.47	5.4	1.84
			52000.	2000.	0.25	6.8	1.81
			60000.	2000.	0.25	6.3	1.76
			75000.	2500.	0.25	5.7	1.70
		25.	44000.	6000.	0.47	8.5	2.40
			52000.	2000.	0.25	10.7	2.35
			60000.	2000.	0.25	10.0	2.27
			75000.	2500.	0.25	8.9	2.18
		50.	44000.	6000.	0.47	12.0	3.02
			52000.	2000.	0.25	15.2	2.96
			60000.	2000.	0.25	14.1	2.84
			75000.	2500.	0.25	12.6	2.72
		75.	44000.	6000.	0.47	14.7	3.51
			52000.	2000.	0.25	18.6	3.43
			60000.	2000.	0.25	17.3	3.28
			75000.	2500.	0.25	15.5	3.13
		100.	44000.	6000.	0.47	17.0	3.91
			52000.	2000.	0.25	21.5	3.82
			60000.	2000.	0.25	20.0	3.65
			75000.	2500.	0.25	17.9	3.48
		150.	44000.	6000.	0.47	20.8	4.59
			52000.	2000.	0.25	26.3	4.48
			60000.	2000.	0.25	24.5	4.28
			75000.	2500.	0.25	21.9	4.06
		200.	44000.	6000.	0.47	24.0	5.17
			52000.	2000.	0.25	30.4	5.04
			60000.	2000.	0.25	28.3	4.80
			75000.	2500.	0.25	25.3	4.56
		250.	44000.	6000.	0.47	26.8	5.67
			52000.	2000.	0.25	34.0	5.53
			60000.	2000.	0.25	31.6	5.27
			75000.	2500.	0.25	28.3	4.99
		300.	44000.	6000.	0.47	29.4	6.13
			52000.	2000.	0.25	37.2	5.97
			60000.	2000.	0.25	34.6	5.69
			75000.	2500.	0.25	31.0	5.38
		350.	44000.	6000.	0.47	31.7	6.55
			52000.	2000.	0.25	40.2	6.38
			60000.	2000.	0.25	37.4	6.07
			75000.	2500.	0.25	33.5	5.74

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\ D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
10.5	0.6	10.	44000.	2000.	0.35	6.5	1.83
			52000.	2000.	0.35	6.0	1.77
			60000.	2500.	0.35	5.6	1.72
			75000.	3000.	0.35	5.0	1.67
		25.	44000.	2000.	0.35	10.3	2.38
			52000.	2000.	0.35	9.5	2.29
			60000.	2500.	0.35	8.8	2.21
			75000.	3000.	0.35	7.9	2.13
		50.	44000.	2000.	0.35	14.6	3.01
			52000.	2000.	0.35	13.4	2.87
			60000.	2500.	0.35	12.5	2.76
			75000.	3000.	0.35	11.2	2.65
		75.	44000.	2000.	0.35	17.8	3.48
			52000.	2000.	0.35	16.4	3.31
			60000.	2500.	0.35	15.3	3.18
			75000.	3000.	0.35	13.7	3.04
		100.	44000.	2000.	0.35	20.6	3.89
			52000.	2000.	0.35	18.9	3.69
			60000.	2500.	0.35	17.6	3.53
			75000.	3000.	0.35	15.8	3.38
		150.	44000.	2000.	0.35	25.2	4.56
			52000.	2000.	0.35	23.2	4.32
			60000.	2500.	0.35	21.6	4.13
			75000.	3000.	0.35	19.3	3.94
		200.	44000.	2000.	0.35	29.1	5.13
			52000.	2000.	0.35	26.8	4.86
			60000.	2500.	0.35	24.9	4.63
			75000.	3000.	0.35	22.3	4.41
		250.	44000.	2000.	0.35	32.6	5.63
			52000.	2000.	0.35	30.0	5.32
			60000.	2500.	0.35	27.9	5.07
			75000.	3000.	0.35	24.9	4.83
		300.	44000.	2000.	0.35	35.7	6.09
			52000.	2000.	0.35	32.8	5.75
			60000.	2500.	0.35	30.6	5.47
			75000.	3000.	0.35	27.3	5.21
		350.	44000.	2000.	0.35	38.5	6.50
			52000.	2000.	0.35	35.4	6.14
			60000.	2500.	0.35	33.0	5.84
			75000.	3000.	0.35	29.5	5.55

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
14.0	1.0	10.	44000.	2000.	0.25	8.6	2.19
			52000.	2000.	0.25	7.9	2.10
			60000.	2500.	0.25	7.3	2.03
			75000.	3000.	0.25	6.6	1.97
		25.	44000.	2000.	0.25	13.5	2.95
			52000.	2000.	0.25	12.5	2.81
			60000.	2500.	0.25	11.6	2.70
			75000.	3000.	0.25	10.4	2.60
		50.	44000.	2000.	0.25	19.1	3.81
			52000.	2000.	0.25	17.6	3.61
			60000.	2500.	0.25	16.4	3.46
			75000.	3000.	0.25	14.7	3.31
		75.	44000.	2000.	0.25	23.5	4.46
			52000.	2000.	0.25	21.6	4.23
			60000.	2500.	0.25	20.1	4.04
			75000.	3000.	0.25	18.0	3.86
		100.	44000.	2000.	0.25	27.1	5.02
			52000.	2000.	0.25	24.9	4.75
			60000.	2500.	0.25	23.2	4.53
			75000.	3000.	0.25	20.7	4.32
		150.	44000.	2000.	0.25	33.2	5.95
			52000.	2000.	0.25	30.5	5.61
			60000.	2500.	0.25	28.4	5.34
			75000.	3000.	0.25	25.4	5.09
		200.	44000.	2000.	0.25	38.3	6.73
			52000.	2000.	0.25	35.2	6.35
			60000.	2500.	0.25	32.8	6.04
			75000.	3000.	0.25	29.3	5.74
		250.	44000.	2000.	0.25	42.8	7.42
			52000.	2000.	0.25	39.4	6.99
			60000.	2500.	0.25	36.7	6.64
			75000.	3000.	0.25	32.8	6.32
		300.	44000.	2000.	0.25	46.9	8.05
			52000.	2000.	0.25	43.1	7.58
			60000.	2500.	0.25	40.2	7.19
			75000.	3000.	0.25	35.9	6.84
		350.	44000.	2000.	0.25	50.7	8.62
			52000.	2000.	0.25	46.6	8.11
			60000.	2500.	0.25	43.4	7.70
			75000.	3000.	0.25	38.8	7.31

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
14.0	0.9	10.	44000.	2000.	0.25	9.0	2.15
			52000.	2000.	0.25	8.3	2.06
			60000.	2000.	0.25	7.7	2.00
			75000.	2500.	0.25	6.9	1.93
		25.	44000.	2000.	0.25	14.2	2.89
			52000.	2000.	0.25	13.1	2.75
			60000.	2000.	0.25	12.2	2.65
			75000.	2500.	0.25	10.9	2.54
		50.	44000.	2000.	0.25	20.1	3.72
			52000.	2000.	0.25	18.5	3.53
			60000.	2000.	0.25	17.2	3.38
			75000.	2500.	0.25	15.4	3.22
		75.	44000.	2000.	0.25	24.6	4.36
			52000.	2000.	0.25	22.6	4.12
			60000.	2000.	0.25	21.1	3.94
			75000.	2500.	0.25	18.8	3.75
		100.	44000.	2000.	0.25	28.4	4.90
			52000.	2000.	0.25	26.1	4.63
			60000.	2000.	0.25	24.3	4.41
			75000.	2500.	0.25	21.7	4.19
		150.	44000.	2000.	0.25	34.8	5.81
			52000.	2000.	0.25	32.0	5.47
			60000.	2000.	0.25	29.8	5.21
			75000.	2500.	0.25	26.6	4.94
		200.	44000.	2000.	0.25	40.1	6.57
			52000.	2000.	0.25	36.9	6.18
			60000.	2000.	0.25	34.4	5.87
			75000.	2500.	0.25	30.7	5.57
		250.	44000.	2000.	0.25	44.9	7.24
			52000.	2000.	0.25	41.3	6.80
			60000.	2000.	0.25	38.4	6.46
			75000.	2500.	0.25	34.4	6.12
		300.	44000.	2000.	0.25	49.2	7.85
			52000.	2000.	0.25	45.2	7.37
			60000.	2000.	0.25	42.1	7.00
			75000.	2500.	0.25	37.7	6.62
		350.	44000.	2000.	0.25	53.1	8.41
			52000.	2000.	0.25	48.9	7.89
			60000.	2000.	0.25	45.5	7.49
			75000.	2500.	0.25	40.7	7.08

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
14.0	0.8	10.	44000.	6000.	0.44	7.1	2.14
			52000.	2000.	0.25	8.6	2.07
			60000.	2000.	0.25	8.0	2.00
			75000.	2000.	0.25	7.2	1.93
		25.	44000.	6000.	0.44	11.2	2.88
			52000.	2000.	0.25	13.7	2.76
			60000.	2000.	0.25	12.7	2.65
			75000.	2000.	0.25	11.4	2.54
		50.	44000.	6000.	0.44	15.8	3.71
			52000.	2000.	0.25	19.3	3.54
			60000.	2000.	0.25	18.0	3.38
			75000.	2000.	0.25	16.1	3.22
		75.	44000.	6000.	0.44	19.4	4.34
			52000.	2000.	0.25	23.7	4.14
			60000.	2000.	0.25	22.0	3.94
			75000.	2000.	0.25	19.7	3.75
		100.	44000.	6000.	0.44	22.4	4.88
			52000.	2000.	0.25	27.3	4.64
			60000.	2000.	0.25	25.5	4.42
			75000.	2000.	0.25	22.8	4.19
		150.	44000.	6000.	0.44	27.4	5.78
			52000.	2000.	0.25	33.5	5.48
			60000.	2000.	0.25	31.2	5.21
			75000.	2000.	0.25	27.9	4.94
		200.	44000.	6000.	0.44	31.6	6.53
			52000.	2000.	0.25	38.7	6.20
			60000.	2000.	0.25	36.0	5.88
			75000.	2000.	0.25	32.2	5.56
		250.	44000.	6000.	0.44	35.4	7.20
			52000.	2000.	0.25	43.2	6.82
			60000.	2000.	0.25	40.2	6.47
			75000.	2000.	0.25	36.0	6.12
		300.	44000.	6000.	0.44	38.7	7.80
			52000.	2000.	0.25	47.4	7.39
			60000.	2000.	0.25	44.1	7.01
			75000.	2000.	0.25	39.4	6.62
		350.	44000.	6000.	0.44	41.8	8.36
			52000.	2000.	0.25	51.2	7.91
			60000.	2000.	0.25	47.6	7.50
			75000.	2000.	0.25	42.6	7.08

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9$ D)

L_s ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
14.0	0.7	10.	44000.	6000.	0.47	7.1	2.15
			52000.	2000.	0.25	9.1	2.09
			60000.	2000.	0.25	8.4	2.02
			75000.	2000.	0.25	7.5	1.94
		25.	44000.	6000.	0.47	11.3	2.88
			52000.	2000.	0.25	14.3	2.80
			60000.	2000.	0.25	13.3	2.68
			75000.	2000.	0.25	11.9	2.56
		50.	44000.	6000.	0.47	16.0	3.71
			52000.	2000.	0.25	20.2	3.59
			60000.	2000.	0.25	18.9	3.43
			75000.	2000.	0.25	16.9	3.26
		75.	44000.	6000.	0.47	19.6	4.35
			52000.	2000.	0.25	24.8	4.20
			60000.	2000.	0.25	23.1	4.00
			75000.	2000.	0.25	20.6	3.80
		100.	44000.	6000.	0.47	22.6	4.89
			52000.	2000.	0.25	28.6	4.72
			60000.	2000.	0.25	26.7	4.49
			75000.	2000.	0.25	23.8	4.25
		150.	44000.	6000.	0.47	27.7	5.79
			52000.	2000.	0.25	35.1	5.58
			60000.	2000.	0.25	32.6	5.30
			75000.	2000.	0.25	29.2	5.00
		200.	44000.	6000.	0.47	32.0	6.55
			52000.	2000.	0.25	40.5	6.30
			60000.	2000.	0.25	37.7	5.98
			75000.	2000.	0.25	33.7	5.64
		250.	44000.	6000.	0.47	35.7	7.21
			52000.	2000.	0.25	45.3	6.94
			60000.	2000.	0.25	42.1	6.58
			75000.	2000.	0.25	37.7	6.20
		300.	44000.	6000.	0.47	39.1	7.82
			52000.	2000.	0.25	49.6	7.52
			60000.	2000.	0.25	46.2	7.13
			75000.	2000.	0.25	41.3	6.71
		350.	44000.	6000.	0.47	42.3	8.38
			52000.	2000.	0.25	53.6	8.06
			60000.	2000.	0.25	49.9	7.63
			75000.	2000.	0.25	44.6	7.18

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = e\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
14.0	0.6	10.	44000.	2000.	0.35	8.7	2.12
			52000.	2000.	0.35	8.0	2.03
			60000.	2000.	0.35	7.4	1.97
			75000.	2500.	0.35	6.7	1.90
		25.	44000.	2000.	0.35	13.7	2.84
			52000.	2000.	0.35	12.6	2.70
			60000.	2000.	0.35	11.8	2.60
			75000.	2500.	0.35	10.5	2.49
		50.	44000.	2000.	0.35	19.4	3.65
			52000.	2000.	0.35	17.9	3.46
			60000.	2000.	0.35	16.6	3.31
			75000.	2500.	0.35	14.9	3.16
		75.	44000.	2000.	0.35	23.8	4.27
			52000.	2000.	0.35	21.9	4.04
			60000.	2000.	0.35	20.4	3.86
			75000.	2500.	0.35	18.2	3.68
		100.	44000.	2000.	0.35	27.5	4.80
			52000.	2000.	0.35	25.3	4.53
			60000.	2000.	0.35	23.5	4.32
			75000.	2500.	0.35	21.0	4.11
		150.	44000.	2000.	0.35	33.6	5.68
			52000.	2000.	0.35	30.9	5.35
			60000.	2000.	0.35	28.8	5.09
			75000.	2500.	0.35	25.8	4.83
		200.	44000.	2000.	0.35	38.8	6.42
			52000.	2000.	0.35	35.7	6.04
			60000.	2000.	0.35	33.3	5.74
			75000.	2500.	0.35	29.7	5.45
		250.	44000.	2000.	0.35	43.4	7.07
			52000.	2000.	0.35	39.9	6.65
			60000.	2000.	0.35	37.2	6.32
			75000.	2500.	0.35	33.3	5.99
		300.	44000.	2000.	0.35	47.6	7.66
			52000.	2000.	0.35	43.8	7.20
			60000.	2000.	0.35	40.7	6.84
			75000.	2500.	0.35	36.4	6.47
		350.	44000.	2000.	0.35	51.4	8.21
			52000.	2000.	0.35	47.3	7.71
			60000.	2000.	0.35	44.0	7.31
			75000.	2500.	0.35	39.4	6.92

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\,D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
17.5	1.0	10.	44000.	2000.	0.25	10.7	2.49
			52000.	2000.	0.25	9.8	2.38
			60000.	2000.	0.25	9.2	2.29
			75000.	2500.	0.25	8.2	2.21
		25.	44000.	2000.	0.25	16.9	3.42
			52000.	2000.	0.25	15.6	3.25
			60000.	2000.	0.25	14.5	3.11
			75000.	2500.	0.25	13.0	2.98
		50.	44000.	2000.	0.25	23.9	4.48
			52000.	2000.	0.25	22.0	4.23
			60000.	2000.	0.25	20.5	4.04
			75000.	2500.	0.25	18.3	3.85
		75.	44000.	2000.	0.25	29.3	5.28
			52000.	2000.	0.25	27.0	4.98
			60000.	2000.	0.25	25.1	4.75
			75000.	2500.	0.25	22.5	4.52
		100.	44000.	2000.	0.25	33.8	5.97
			52000.	2000.	0.25	31.1	5.62
			60000.	2000.	0.25	29.0	5.34
			75000.	2500.	0.25	25.9	5.08
		150.	44000.	2000.	0.25	41.5	7.11
			52000.	2000.	0.25	38.1	6.68
			60000.	2000.	0.25	35.5	6.35
			75000.	2500.	0.25	31.8	6.02
		200.	44000.	2000.	0.25	47.9	8.07
			52000.	2000.	0.25	44.0	7.58
			60000.	2000.	0.25	41.0	7.19
			75000.	2500.	0.25	36.7	6.82
		250.	44000.	2000.	0.25	53.5	8.92
			52000.	2000.	0.25	49.2	8.37
			60000.	2000.	0.25	45.8	7.94
			75000.	2500.	0.25	41.0	7.52
		300.	44000.	2000.	0.25	58.6	9.69
			52000.	2000.	0.25	53.9	9.08
			60000.	2000.	0.25	50.2	8.61
			75000.	2500.	0.25	44.9	8.15
		350.	44000.	2000.	0.25	63.3	10.39
			52000.	2000.	0.25	58.3	9.74
			60000.	2000.	0.25	54.2	9.23
			75000.	2500.	0.25	48.5	8.73

TABLE 3-40 (Cont'd)

MINIMUM IN - PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	$\bar{\phi}_{Sc}$ psi	D in.	Ct $/sq ft
17.5	0.9	10.	44000.	2000.	0.25	11.2	2.45
			52000.	2000.	0.25	10.3	2.34
			60000.	2000.	0.25	9.6	2.25
			75000.	2000.	0.25	8.6	2.16
		25.	44000.	2000.	0.25	17.7	3.36
			52000.	2000.	0.25	16.3	3.18
			60000.	2000.	0.25	15.2	3.05
			75000.	2000.	0.25	13.6	2.91
		50.	44000.	2000.	0.25	25.1	4.39
			52000.	2000.	0.25	23.1	4.14
			60000.	2000.	0.25	21.5	3.94
			75000.	2000.	0.25	19.2	3.75
		75.	44000.	2000.	0.25	30.7	5.17
			52000.	2000.	0.25	28.3	4.87
			60000.	2000.	0.25	26.3	4.63
			75000.	2000.	0.25	23.5	4.39
		100.	44000.	2000.	0.25	35.5	5.84
			52000.	2000.	0.25	32.6	5.49
			60000.	2000.	0.25	30.4	5.21
			75000.	2000.	0.25	27.2	4.94
		150.	44000.	2000.	0.25	43.5	6.95
			52000.	2000.	0.25	40.0	6.52
			60000.	2000.	0.25	37.2	6.19
			75000.	2000.	0.25	33.3	5.85
		200.	44000.	2000.	0.25	50.2	7.89
			52000.	2000.	0.25	46.2	7.40
			60000.	2000.	0.25	43.0	7.01
			75000.	2000.	0.25	38.4	6.62
		250.	44000.	2000.	0.25	56.1	8.72
			52000.	2000.	0.25	51.6	8.17
			60000.	2000.	0.25	48.0	7.73
			75000.	2000.	0.25	43.0	7.29
		300.	44000.	2000.	0.25	61.5	9.47
			52000.	2000.	0.25	56.5	8.86
			60000.	2000.	0.25	52.6	8.39
			75000.	2000.	0.25	47.1	7.91
		350.	44000.	2000.	0.25	66.4	10.16
			52000.	2000.	0.25	61.1	9.50
			60000.	2000.	0.25	56.8	8.99
			75000.	2000.	0.25	50.8	8.47

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\,D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
17.5	0.8	10.	44000.	6000.	0.44	8.8	2.45
			52000.	2000.	0.25	10.8	2.34
			60000.	2000.	0.25	10.1	2.26
			75000.	2000.	0.25	9.0	2.16
		25.	44000.	6000.	0.44	14.0	3.36
			52000.	2000.	0.25	17.1	3.20
			60000.	2000.	0.25	15.9	3.05
			75000.	2000.	0.25	14.2	2.91
		50.	44000.	6000.	0.44	19.8	4.39
			52000.	2000.	0.25	24.2	4.15
			60000.	2000.	0.25	22.5	3.96
			75000.	2000.	0.25	20.1	3.75
		75.	44000.	6000.	0.44	24.2	5.18
			52000.	2000.	0.25	29.6	4.89
			60000.	2000.	0.25	27.6	4.65
			75000.	2000.	0.25	24.6	4.39
		100.	44000.	6000.	0.44	28.0	5.85
			52000.	2000.	0.25	34.2	5.51
			60000.	2000.	0.25	31.8	5.23
			75000.	2000.	0.25	28.5	4.93
		150.	44000.	6000.	0.44	34.2	6.97
			52000.	2000.	0.25	41.9	6.55
			60000.	2000.	0.25	39.0	6.21
			75000.	2000.	0.25	34.9	5.84
		200.	44000.	6000.	0.44	39.5	7.91
			52000.	2000.	0.25	48.3	7.43
			60000.	2000.	0.25	45.0	7.03
			75000.	2000.	0.25	40.2	6.61
		250.	44000.	6000.	0.44	44.2	8.74
			52000.	2000.	0.25	54.0	8.20
			60000.	2000.	0.25	50.3	7.76
			75000.	2000.	0.25	45.0	7.29
		300.	44000.	6000.	0.44	48.4	9.49
			52000.	2000.	0.25	59.2	8.90
			60000.	2000.	0.25	55.1	8.41
			75000.	2000.	0.25	49.3	7.90
		350.	44000.	6000.	0.44	52.3	10.18
			52000.	2000.	0.25	63.9	9.54
			60000.	2000.	0.25	59.5	9.02
			75000.	2000.	0.25	53.2	8.46

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9$ D)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
17.5	0.7	10.	44000.	6000.	0.47	8.9	2.46
			52000.	2000.	0.25	11.3	2.38
			60000.	2000.	0.25	10.5	2.28
			75000.	2000.	0.25	9.4	2.18
		25.	44000.	6000.	0.47	14.1	3.37
			52000.	2000.	0.25	17.9	3.25
			60000.	2000.	0.25	16.7	3.10
			75000.	2000.	0.25	14.9	2.94
		50.	44000.	6000.	0.47	20.0	4.40
			52000.	2000.	0.25	25.3	4.23
			60000.	2000.	0.25	23.6	4.02
			75000.	2000.	0.25	21.1	3.80
		75.	44000.	6000.	0.47	24.5	5.19
			52000.	2000.	0.25	31.0	4.98
			60000.	2000.	0.25	28.9	4.73
			75000.	2000.	0.25	25.8	4.45
		100.	44000.	6000.	0.47	28.3	5.86
			52000.	2000.	0.25	35.8	5.61
			60000.	2000.	0.25	33.3	5.32
			75000.	2000.	0.25	29.8	5.01
		150.	44000.	6000.	0.47	34.6	6.98
			52000.	2000.	0.25	43.8	6.67
			60000.	2000.	0.25	40.8	6.32
			75000.	2000.	0.25	36.5	5.93
		200.	44000.	6000.	0.47	40.0	7.92
			52000.	2000.	0.25	50.6	7.57
			60000.	2000.	0.25	47.1	7.16
			75000.	2000.	0.25	42.1	6.71
		250.	44000.	6000.	0.47	44.7	8.76
			52000.	2000.	0.25	56.6	8.36
			60000.	2000.	0.25	52.7	7.90
			75000.	2000.	0.25	47.1	7.40
		300.	44000.	6000.	0.47	48.9	9.51
			52000.	2000.	0.25	62.0	9.07
			60000.	2000.	0.25	57.7	8.57
			75000.	2000.	0.25	51.6	8.02
		350.	44000.	6000.	0.47	52.9	10.20
			52000.	2000.	0.25	67.0	9.73
			60000.	2000.	0.25	62.3	9.19
			75000.	2000.	0.25	55.8	8.60

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\ D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
17.5	0.6	10.	44000.	2000.	0.35	10.9	2.41
			52000.	2000.	0.35	10.0	2.30
			60000.	2000.	0.35	9.3	2.21
			75000.	2000.	0.35	8.3	2.13
		25.	44000.	2000.	0.35	17.2	3.29
			52000.	2000.	0.35	15.8	3.12
			60000.	2000.	0.35	14.7	2.99
			75000.	2000.	0.35	13.1	2.86
		50.	44000.	2000.	0.35	24.3	4.29
			52000.	2000.	0.35	22.3	4.05
			60000.	2000.	0.35	20.8	3.86
			75000.	2000.	0.35	18.6	3.67
		75.	44000.	2000.	0.35	29.7	5.06
			52000.	2000.	0.35	27.3	4.77
			60000.	2000.	0.35	25.5	4.53
			75000.	2000.	0.35	22.8	4.30
		100.	44000.	2000.	0.35	34.3	5.71
			52000.	2000.	0.35	31.6	5.37
			60000.	2000.	0.35	29.4	5.10
			75000.	2000.	0.35	26.3	4.83
		150.	44000.	2000.	0.35	42.0	6.79
			52000.	2000.	0.35	38.7	6.38
			60000.	2000.	0.35	36.0	6.05
			75000.	2000.	0.35	32.2	5.72
		200.	44000.	2000.	0.35	48.6	7.71
			52000.	2000.	0.35	44.7	7.23
			60000.	2000.	0.35	41.6	6.85
			75000.	2000.	0.35	37.2	6.47
		250.	44000.	2000.	0.35	54.3	8.51
			52000.	2000.	0.35	49.9	7.97
			60000.	2000.	0.35	46.5	7.55
			75000.	2000.	0.35	41.6	7.13
		300.	44000.	2000.	0.35	59.5	9.24
			52000.	2000.	0.35	54.7	8.65
			60000.	2000.	0.35	50.9	8.19
			75000.	2000.	0.35	45.5	7.73
		350.	44000.	2000.	0.35	64.2	9.91
			52000.	2000.	0.35	59.1	9.27
			60000.	2000.	0.35	55.0	8.77
			75000.	2000.	0.35	49.2	8.27

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS, WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
21.0	1.0	10.	44000.	2000.	0.25	12.8	2.79
			52000.	2000.	0.25	11.8	2.65
			60000.	2000.	0.25	11.0	2.55
			75000.	2500.	0.25	9.8	2.45
		25.	44000.	2000.	0.25	20.3	3.90
			52000.	2000.	0.25	18.7	3.68
			60000.	2000.	0.25	17.4	3.52
			75000.	2500.	0.25	15.6	3.36
		50.	44000.	2000.	0.25	28.7	5.15
			52000.	2000.	0.25	26.4	4.85
			60000.	2000.	0.25	24.6	4.61
			75000.	2500.	0.25	22.0	4.39
		75.	44000.	2000.	0.25	35.2	6.10
			52000.	2000.	0.25	32.4	5.74
			60000.	2000.	0.25	30.1	5.45
			75000.	2500.	0.25	26.9	5.17
		100.	44000.	2000.	0.25	40.6	6.91
			52000.	2000.	0.25	37.4	6.49
			60000.	2000.	0.25	34.8	6.16
			75000.	2500.	0.25	31.1	5.84
		150.	44000.	2000.	0.25	49.7	8.27
			52000.	2000.	0.25	45.8	7.75
			60000.	2000.	0.25	42.6	7.34
			75000.	2500.	0.25	38.1	6.95
		200.	44000.	2000.	0.25	57.4	9.41
			52000.	2000.	0.25	52.8	8.81
			60000.	2000.	0.25	49.2	8.34
			75000.	2500.	0.25	44.0	7.89
		250.	44000.	2000.	0.25	64.2	10.42
			52000.	2000.	0.25	59.1	9.75
			60000.	2000.	0.25	55.0	9.22
			75000.	2500.	0.25	49.2	8.72
		300.	44000.	2000.	0.25	70.4	11.33
			52000.	2000.	0.25	64.7	10.59
			60000.	2000.	0.25	60.2	10.02
			75000.	2500.	0.25	53.9	9.47
		350.	44000.	2000.	0.25	76.0	12.17
			52000.	2000.	0.25	69.9	11.37
			60000.	2000.	0.25	65.1	10.75
			75000.	2500.	0.25	58.2	10.16

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9$ D)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
21.0	0.9	10.	44000.	2000.	0.25	13.5	2.74
			52000.	2000.	0.25	12.4	2.61
			60000.	2000.	0.25	11.5	2.50
			75000.	2000.	0.25	10.3	2.39
		25.	44000.	2000.	0.25	21.3	3.83
			52000.	2000.	0.25	19.6	3.61
			60000.	2000.	0.25	18.2	3.45
			75000.	2000.	0.25	16.3	3.27
		50.	44000.	2000.	0.25	30.1	5.05
			52000.	2000.	0.25	27.7	4.75
			60000.	2000.	0.25	25.8	4.51
			75000.	2000.	0.25	23.1	4.27
		75.	44000.	2000.	0.25	36.9	5.98
			52000.	2000.	0.25	33.9	5.62
			60000.	2000.	0.25	31.6	5.33
			75000.	2000.	0.25	28.2	5.03
		100.	44000.	2000.	0.25	42.6	6.77
			52000.	2000.	0.25	39.2	6.35
			60000.	2000.	0.25	36.5	6.01
			75000.	2000.	0.25	32.6	5.67
		150.	44000.	2000.	0.25	52.1	8.10
			52000.	2000.	0.25	48.0	7.58
			60000.	2000.	0.25	44.7	7.17
			75000.	2000.	0.25	39.9	6.74
		200.	44000.	2000.	0.25	60.2	9.21
			52000.	2000.	0.25	55.4	8.61
			60000.	2000.	0.25	51.6	8.14
			75000.	2000.	0.25	46.1	7.65
		250.	44000.	2000.	0.25	67.3	10.20
			52000.	2000.	0.25	61.9	9.53
			60000.	2000.	0.25	57.7	9.00
			75000.	2000.	0.25	51.6	8.45
		300.	44000.	2000.	0.25	73.7	11.09
			52000.	2000.	0.25	67.8	10.35
			60000.	2000.	0.25	63.2	9.77
			75000.	2000.	0.25	56.5	9.17
		350.	44000.	2000.	0.25	79.7	11.91
			52000.	2000.	0.25	73.3	11.11
			60000.	2000.	0.25	68.2	10.49
			75000.	2000.	0.25	61.0	9.84

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
21.0	0.8	10.	44000.	2000.	0.25	14.1	2.76
			52000.	2000.	0.25	13.0	2.62
			60000.	2000.	0.25	12.1	2.51
			75000.	2000.	0.25	10.8	2.40
		25.	44000.	2000.	0.25	22.3	3.85
			52000.	2000.	0.25	20.5	3.63
			60000.	2000.	0.25	19.1	3.46
			75000.	2000.	0.25	17.1	3.28
		50.	44000.	2000.	0.25	31.5	5.08
			52000.	2000.	0.25	29.0	4.77
			60000.	2000.	0.25	27.0	4.53
			75000.	2000.	0.25	24.1	4.27
		75.	44000.	2000.	0.25	38.6	6.02
			52000.	2000.	0.25	35.5	5.65
			60000.	2000.	0.25	33.1	5.35
			75000.	2000.	0.25	29.6	5.03
		100.	44000.	2000.	0.25	44.6	6.82
			52000.	2000.	0.25	41.0	6.38
			60000.	2000.	0.25	38.2	6.04
			75000.	2000.	0.25	34.1	5.67
		150.	44000.	2000.	0.25	54.6	8.15
			52000.	2000.	0.25	50.2	7.62
			60000.	2000.	0.25	46.8	7.20
			75000.	2000.	0.25	41.8	6.75
		200.	44000.	2000.	0.25	63.1	9.28
			52000.	2000.	0.25	58.0	8.66
			60000.	2000.	0.25	54.0	8.18
			75000.	2000.	0.25	48.3	7.66
		250.	44000.	2000.	0.25	70.5	10.27
			52000.	2000.	0.25	64.8	9.58
			60000.	2000.	0.25	60.4	9.04
			75000.	2000.	0.25	54.0	8.46
		300.	44000.	2000.	0.25	77.2	11.16
			52000.	2000.	0.25	71.0	10.41
			60000.	2000.	0.25	66.1	9.82
			75000.	2000.	0.25	59.1	9.18
		350.	44000.	2000.	0.25	83.4	11.99
			52000.	2000.	0.25	76.7	11.17
			60000.	2000.	0.25	71.4	10.54
			75000.	2000.	0.25	63.9	9.85

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\,D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
21.0	0.7	10.	44000.	6000.	0.47	10.7	2.76
			52000.	2000.	0.25	13.6	2.66
			60000.	2000.	0.25	12.6	2.55
			75000.	2000.	0.25	11.3	2.42
		25.	44000.	6000.	0.47	17.0	3.86
			52000.	2000.	0.25	21.5	3.69
			60000.	2000.	0.25	20.0	3.52
			75000.	2000.	0.25	17.9	3.32
		50.	44000.	6000.	0.47	24.0	5.09
			52000.	2000.	0.25	30.4	4.86
			60000.	2000.	0.25	28.3	4.61
			75000.	2000.	0.25	25.3	4.33
		75.	44000.	6000.	0.47	29.4	6.04
			52000.	2000.	0.25	37.2	5.75
			60000.	2000.	0.25	34.6	5.45
			75000.	2000.	0.25	31.0	5.11
		100.	44000.	6000.	0.47	33.9	6.84
			52000.	2000.	0.25	43.0	6.51
			60000.	2000.	0.25	40.0	6.15
			75000.	2000.	0.25	35.8	5.76
		150.	44000.	6000.	0.47	41.5	8.17
			52000.	2000.	0.25	52.6	7.77
			60000.	2000.	0.25	49.0	7.34
			75000.	2000.	0.25	43.8	6.86
		200.	44000.	6000.	0.47	47.9	9.30
			52000.	2000.	0.25	60.7	8.84
			60000.	2000.	0.25	56.6	8.34
			75000.	2000.	0.25	50.6	7.79
		250.	44000.	6000.	0.47	53.6	10.30
			52000.	2000.	0.25	67.9	9.78
			60000.	2000.	0.25	63.2	9.22
			75000.	2000.	0.25	56.6	8.60
		300.	44000.	6000.	0.47	58.7	11.20
			52000.	2000.	0.25	74.4	10.62
			60000.	2000.	0.25	69.3	10.01
			75000.	2000.	0.25	61.9	9.34
		350.	44000.	6000.	0.47	63.4	12.02
			52000.	2000.	0.25	80.4	11.40
			60000.	2000.	0.25	74.8	10.75
			75000.	2000.	0.25	66.9	10.02

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, d = 0.9 D)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
21.0	0.6	10.	44000.	2000.	0.35	13.0	2.69
			52000.	2000.	0.35	12.0	2.56
			60000.	2000.	0.35	11.2	2.46
			75000.	2000.	0.35	10.0	2.36
		25.	44000.	2000.	0.35	20.6	3.75
			52000.	2000.	0.35	18.9	3.54
			60000.	2000.	0.35	17.6	3.38
			75000.	2000.	0.35	15.8	3.21
		50.	44000.	2000.	0.35	29.1	4.94
			52000.	2000.	0.35	26.8	4.65
			60000.	2000.	0.35	24.9	4.42
			75000.	2000.	0.35	22.3	4.18
		75.	44000.	2000.	0.35	35.7	5.85
			52000.	2000.	0.35	32.8	5.49
			60000.	2000.	0.35	30.6	5.21
			75000.	2000.	0.35	27.3	4.92
		100.	44000.	2000.	0.35	41.2	6.62
			52000.	2000.	0.35	37.9	6.20
			60000.	2000.	0.35	35.3	5.88
			75000.	2000.	0.35	31.6	5.54
		150.	44000.	2000.	0.35	50.5	7.91
			52000.	2000.	0.35	46.4	7.40
			60000.	2000.	0.35	43.2	7.00
			75000.	2000.	0.35	38.6	6.59
		200.	44000.	2000.	0.35	58.3	9.00
			52000.	2000.	0.35	53.6	8.41
			60000.	2000.	0.35	49.9	7.95
			75000.	2000.	0.35	44.6	7.48
		250.	44000.	2000.	0.35	65.1	9.95
			52000.	2000.	0.35	59.9	9.30
			60000.	2000.	0.35	55.8	8.79
			75000.	2000.	0.35	49.9	8.26
		300.	44000.	2000.	0.35	71.4	10.82
			52000.	2000.	0.35	65.6	10.10
			60000.	2000.	0.35	61.1	9.54
			75000.	2000.	0.35	54.7	8.96
		350.	44000.	2000.	0.35	77.1	11.62
			52000.	2000.	0.35	70.9	10.84
			60000.	2000.	0.35	66.0	10.23
			75000.	2000.	0.35	59.0	9.61

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9$ D)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
24.5	1.0	10.	44000.	2000.	0.25	15.0	3.09
			52000.	2000.	0.25	13.8	2.93
			60000.	2000.	0.25	12.8	2.80
			75000.	2000.	0.25	11.5	2.68
		25.	44000.	2000.	0.25	23.7	4.37
			52000.	2000.	0.25	21.8	4.12
			60000.	2000.	0.25	20.3	3.92
			75000.	2000.	0.25	18.1	3.73
		50.	44000.	2000.	0.25	33.5	5.82
			52000.	2000.	0.25	30.8	5.46
			60000.	2000.	0.25	28.7	5.18
			75000.	2000.	0.25	25.7	4.91
		75.	44000.	2000.	0.25	41.0	6.93
			52000.	2000.	0.25	37.8	6.49
			60000.	2000.	0.25	35.1	6.15
			75000.	2000.	0.25	31.4	5.82
		100.	44000.	2000.	0.25	47.4	7.86
			52000.	2000.	0.25	43.6	7.36
			60000.	2000.	0.25	40.6	6.97
			75000.	2000.	0.25	36.3	6.58
		150.	44000.	2000.	0.25	58.0	9.43
			52000.	2000.	0.25	53.4	8.82
			60000.	2000.	0.25	49.7	8.33
			75000.	2000.	0.25	44.5	7.86
		200.	44000.	2000.	0.25	67.0	10.75
			52000.	2000.	0.25	61.6	10.04
			60000.	2000.	0.25	57.4	9.49
			75000.	2000.	0.25	51.3	8.94
		250.	44000.	2000.	0.25	74.9	11.92
			52000.	2000.	0.25	68.9	11.12
			60000.	2000.	0.25	64.2	10.50
			75000.	2000.	0.25	57.4	9.90
		300.	44000.	2000.	0.25	82.1	12.97
			52000.	2000.	0.25	75.5	12.10
			60000.	2000.	0.25	70.3	11.42
			75000.	2000.	0.25	62.9	10.76
		350.	44000.	2000.	0.25	88.7	13.94
			52000.	2000.	0.25	81.6	13.00
			60000.	2000.	0.25	75.9	12.27
			75000.	2000.	0.25	67.9	11.55

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\ D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
24.5	0.9	10.	44000.	2000.	0.25	15.7	3.04
			52000.	2000.	0.25	14.5	2.88
			60000.	2000.	0.25	13.5	2.76
			75000.	2000.	0.25	12.0	2.63
		25.	44000.	2000.	0.25	24.8	4.29
			52000.	2000.	0.25	22.8	4.04
			60000.	2000.	0.25	21.3	3.85
			75000.	2000.	0.25	19.0	3.64
		50.	44000.	2000.	0.25	35.1	5.71
			52000.	2000.	0.25	32.3	5.36
			60000.	2000.	0.25	30.1	5.08
			75000.	2000.	0.25	26.9	4.78
		75.	44000.	2000.	0.25	43.0	6.79
			52000.	2000.	0.25	39.6	6.36
			60000.	2000.	0.25	36.8	6.02
			75000.	2000.	0.25	33.0	5.66
		100.	44000.	2000.	0.25	49.7	7.71
			52000.	2000.	0.25	45.7	7.21
			60000.	2000.	0.25	42.5	6.82
			75000.	2000.	0.25	38.0	6.40
		150.	44000.	2000.	0.25	60.8	9.24
			52000.	2000.	0.25	56.0	8.63
			60000.	2000.	0.25	52.1	8.15
			75000.	2000.	0.25	46.6	7.64
		200.	44000.	2000.	0.25	70.3	10.54
			52000.	2000.	0.25	64.6	9.83
			60000.	2000.	0.25	60.2	9.27
			75000.	2000.	0.25	53.8	8.69
		250.	44000.	2000.	0.25	78.5	11.68
			52000.	2000.	0.25	72.3	10.89
			60000.	2000.	0.25	67.3	10.26
			75000.	2000.	0.25	60.2	9.61
		300.	44000.	2000.	0.25	86.0	12.71
			52000.	2000.	0.25	79.1	11.84
			60000.	2000.	0.25	73.7	11.16
			75000.	2000.	0.25	65.9	10.44
		350.	44000.	2000.	0.25	92.9	13.66
			52000.	2000.	0.25	85.5	12.72
			60000.	2000.	0.25	79.6	11.98
			75000.	2000.	0.25	71.2	11.21

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9$ D)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
24.5	0.8	10.	44000.	2000.	0.25	16.4	3.06
			52000.	2000.	0.25	15.1	2.90
			60000.	2000.	0.25	14.1	2.77
			75000.	2000.	0.25	12.6	2.63
		25.	44000.	2000.	0.25	26.0	4.32
			52000.	2000.	0.25	23.9	4.07
			60000.	2000.	0.25	22.3	3.87
			75000.	2000.	0.25	19.9	3.65
		50.	44000.	2000.	0.25	36.8	5.75
			52000.	2000.	0.25	33.8	5.39
			60000.	2000.	0.25	31.5	5.10
			75000.	2000.	0.25	28.2	4.79
		75.	44000.	2000.	0.25	45.0	6.84
			52000.	2000.	0.25	41.4	6.40
			60000.	2000.	0.25	38.6	6.05
			75000.	2000.	0.25	34.5	5.67
		100.	44000.	2000.	0.25	52.0	7.76
			52000.	2000.	0.25	47.8	7.25
			60000.	2000.	0.25	44.5	6.85
			75000.	2000.	0.25	39.8	6.41
		150.	44000.	2000.	0.25	63.7	9.31
			52000.	2000.	0.25	58.6	8.69
			60000.	2000.	0.25	54.6	8.20
			75000.	2000.	0.25	48.8	7.66
		200.	44000.	2000.	0.25	73.6	10.62
			52000.	2000.	0.25	67.7	9.90
			60000.	2000.	0.25	63.0	9.33
			75000.	2000.	0.25	56.3	8.70
		250.	44000.	2000.	0.25	82.2	11.77
			52000.	2000.	0.25	75.7	10.96
			60000.	2000.	0.25	70.4	10.32
			75000.	2000.	0.25	63.0	9.63
		300.	44000.	2000.	0.25	90.1	12.80
			52000.	2000.	0.25	82.9	11.92
			60000.	2000.	0.25	77.2	11.23
			75000.	2000.	0.25	69.0	10.46
		350.	44000.	2000.	0.25	97.3	13.76
			52000.	2000.	0.25	89.5	12.81
			60000.	2000.	0.25	83.3	12.05
			75000.	2000.	0.25	74.5	11.23

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

$(0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\,D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
24.5	0.7	10.	44000.	6000.	0.47	12.5	3.07
			52000.	2000.	0.25	15.8	2.94
			60000.	2000.	0.25	14.8	2.81
			75000.	2000.	0.25	13.2	2.66
		25.	44000.	6000.	0.47	19.8	4.35
			52000.	2000.	0.25	25.1	4.14
			60000.	2000.	0.25	23.3	3.93
			75000.	2000.	0.25	20.9	3.70
		50.	44000.	6000.	0.47	28.0	5.78
			52000.	2000.	0.25	35.4	5.49
			60000.	2000.	0.25	33.0	5.20
			75000.	2000.	0.25	29.5	4.87
		75.	44000.	6000.	0.47	34.3	6.88
			52000.	2000.	0.25	43.4	6.53
			60000.	2000.	0.25	40.4	6.17
			75000.	2000.	0.25	36.1	5.77
		100.	44000.	6000.	0.47	39.6	7.81
			52000.	2000.	0.25	50.1	7.40
			60000.	2000.	0.25	46.7	6.99
			75000.	2000.	0.25	41.7	6.52
		150.	44000.	6000.	0.47	48.4	9.37
			52000.	2000.	0.25	61.4	8.87
			60000.	2000.	0.25	57.1	8.36
			75000.	2000.	0.25	51.1	7.79
		200.	44000.	6000.	0.47	55.9	10.68
			52000.	2000.	0.25	70.9	10.10
			60000.	2000.	0.25	66.0	9.52
			75000.	2000.	0.25	59.0	8.86
		250.	44000.	6000.	0.47	62.5	11.84
			52000.	2000.	0.25	79.2	11.19
			60000.	2000.	0.25	73.8	10.54
			75000.	2000.	0.25	66.0	9.80
		300.	44000.	6000.	0.47	68.5	12.88
			52000.	2000.	0.25	86.8	12.17
			60000.	2000.	0.25	80.8	11.46
			75000.	2000.	0.25	72.3	10.65
		350.	44000.	6000.	0.47	74.0	13.85
			52000.	2000.	0.25	93.8	13.08
			60000.	2000.	0.25	87.3	12.31
			75000.	2000.	0.25	78.1	11.43

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\,D$)

L_s ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
24.5	0.6	10.	44000.	2000.	0.35	15.2	2.98
			52000.	2000.	0.35	14.0	2.83
			60000.	2000.	0.35	13.0	2.71
			75000.	2000.	0.35	11.6	2.58
		25.	44000.	2000.	0.35	24.0	4.20
			52000.	2000.	0.35	22.1	3.96
			60000.	2000.	0.35	20.6	3.77
			75000.	2000.	0.35	18.4	3.57
		50.	44000.	2000.	0.35	34.0	5.58
			52000.	2000.	0.35	31.3	5.24
			60000.	2000.	0.35	29.1	4.97
			75000.	2000.	0.35	26.0	4.68
		75.	44000.	2000.	0.35	41.6	6.64
			52000.	2000.	0.35	38.3	6.22
			60000.	2000.	0.35	35.6	5.89
			75000.	2000.	0.35	31.9	5.54
		100.	44000.	2000.	0.35	48.1	7.53
			52000.	2000.	0.35	44.2	7.04
			60000.	2000.	0.35	41.2	6.66
			75000.	2000.	0.35	36.8	6.26
		150.	44000.	2000.	0.35	58.9	9.02
			52000.	2000.	0.35	54.1	8.43
			60000.	2000.	0.35	50.4	7.96
			75000.	2000.	0.35	45.1	7.47
		200.	44000.	2000.	0.35	68.0	10.28
			52000.	2000.	0.35	62.5	9.60
			60000.	2000.	0.35	58.2	9.05
			75000.	2000.	0.35	52.1	8.49
		250.	44000.	2000.	0.35	76.0	11.39
			52000.	2000.	0.35	69.9	10.62
			60000.	2000.	0.35	65.1	10.02
			75000.	2000.	0.35	58.2	9.38
		300.	44000.	2000.	0.35	83.2	12.40
			52000.	2000.	0.35	76.6	11.55
			60000.	2000.	0.35	71.3	10.89
			75000.	2000.	0.35	63.8	10.19
		350.	44000.	2000.	0.35	89.9	13.32
			52000.	2000.	0.35	82.7	12.41
			60000.	2000.	0.35	77.0	11.69
			75000.	2000.	0.35	68.9	10.94

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEDMENT

$(0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\ D)$

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
28.0	1.0	10.	44000.	2000.	0.25	17.1	3.39
			52000.	2000.	0.25	15.8	3.20
			60000.	2000.	0.25	14.7	3.06
			75000.	2000.	0.25	13.1	2.92
		25.	44000.	2000.	0.25	27.1	4.84
			52000.	2000.	0.25	24.9	4.56
			60000.	2000.	0.25	23.2	4.33
			75000.	2000.	0.25	20.7	4.10
		50.	44000.	2000.	0.25	38.3	6.49
			52000.	2000.	0.25	35.2	6.08
			60000.	2000.	0.25	32.8	5.76
			75000.	2000.	0.25	29.3	5.44
		75.	44000.	2000.	0.25	46.9	7.75
			52000.	2000.	0.25	43.1	7.25
			60000.	2000.	0.25	40.2	6.85
			75000.	2000.	0.25	35.9	6.46
		100.	44000.	2000.	0.25	54.2	8.81
			52000.	2000.	0.25	49.8	8.23
			60000.	2000.	0.25	46.4	7.78
			75000.	2000.	0.25	41.5	7.33
		150.	44000.	2000.	0.25	66.3	10.59
			52000.	2000.	0.25	61.0	9.88
			60000.	2000.	0.25	56.8	9.33
			75000.	2000.	0.25	50.8	8.77
		200.	44000.	2000.	0.25	76.6	12.09
			52000.	2000.	0.25	70.5	11.28
			60000.	2000.	0.25	65.6	10.63
			75000.	2000.	0.25	58.7	10.00
		250.	44000.	2000.	0.25	85.6	13.41
			52000.	2000.	0.25	78.8	12.50
			60000.	2000.	0.25	73.3	11.79
			75000.	2000.	0.25	65.6	11.07
		300.	44000.	2000.	0.25	93.8	14.61
			52000.	2000.	0.25	86.3	13.61
			60000.	2000.	0.25	80.3	12.83
			75000.	2000.	0.25	71.8	12.05
		350.	44000.	2000.	0.25	101.3	15.71
			52000.	2000.	0.25	93.2	14.63
			60000.	2000.	0.25	86.8	13.78
			75000.	2000.	0.25	77.6	12.94

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00,\ \phi_v = 0,\ 2000 \leq f'_c \leq 6000,\ d = 0.9\,D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
28.0	0.9	10.	44000.	2000.	0.25	18.0	3.34
			52000.	2000.	0.25	16.5	3.15
			60000.	2000.	0.25	15.4	3.01
			75000.	2000.	0.25	13.8	2.86
		25.	44000.	2000.	0.25	28.4	4.76
			52000.	2000.	0.25	26.1	4.47
			60000.	2000.	0.25	24.3	4.25
			75000.	2000.	0.25	21.7	4.01
		50.	44000.	2000.	0.25	40.1	6.37
			52000.	2000.	0.25	36.9	5.96
			60000.	2000.	0.25	34.4	5.64
			75000.	2000.	0.25	30.7	5.30
		75.	44000.	2000.	0.25	49.2	7.60
			52000.	2000.	0.25	45.2	7.11
			60000.	2000.	0.25	42.1	6.71
			75000.	2000.	0.25	37.7	6.29
		100.	44000.	2000.	0.25	56.8	8.64
			52000.	2000.	0.25	52.2	8.07
			60000.	2000.	0.25	48.6	7.62
			75000.	2000.	0.25	43.5	7.13
		150.	44000.	2000.	0.25	69.5	10.39
			52000.	2000.	0.25	64.0	9.68
			60000.	2000.	0.25	59.5	9.13
			75000.	2000.	0.25	53.3	8.54
		200.	44000.	2000.	0.25	80.3	11.86
			52000.	2000.	0.25	73.9	11.05
			60000.	2000.	0.25	68.8	10.41
			75000.	2000.	0.25	61.5	9.72
		250.	44000.	2000.	0.25	89.8	13.16
			52000.	2000.	0.25	82.6	12.25
			60000.	2000.	0.25	76.9	11.53
			75000.	2000.	0.25	68.8	10.76
		300.	44000.	2000.	0.25	98.3	14.33
			52000.	2000.	0.25	90.5	13.33
			60000.	2000.	0.25	84.2	12.55
			75000.	2000.	0.25	75.3	11.71
		350.	44000.	2000.	0.25	106.2	15.41
			52000.	2000.	0.25	97.7	14.33
			60000.	2000.	0.25	91.0	13.48
			75000.	2000.	0.25	81.4	12.58

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\,D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
28.0	0.8	10.	44000.	2000.	0.25	18.8	3.36
			52000.	2000.	0.25	17.3	3.17
			60000.	2000.	0.25	16.1	3.03
			75000.	2000.	0.25	14.4	2.86
		25.	44000.	2000.	0.25	29.7	4.80
			52000.	2000.	0.25	27.3	4.50
			60000.	2000.	0.25	25.5	4.27
			75000.	2000.	0.25	22.8	4.02
		50.	44000.	2000.	0.25	42.0	6.42
			52000.	2000.	0.25	38.7	6.00
			60000.	2000.	0.25	36.0	5.68
			75000.	2000.	0.25	32.2	5.31
		75.	44000.	2000.	0.25	51.5	7.66
			52000.	2000.	0.25	47.4	7.16
			60000.	2000.	0.25	44.1	6.76
			75000.	2000.	0.25	39.4	6.31
		100.	44000.	2000.	0.25	59.4	8.71
			52000.	2000.	0.25	54.7	8.13
			60000.	2000.	0.25	50.9	7.67
			75000.	2000.	0.25	45.5	7.15
		150.	44000.	2000.	0.25	72.8	10.47
			52000.	2000.	0.25	67.0	9.76
			60000.	2000.	0.25	62.3	9.19
			75000.	2000.	0.25	55.8	8.56
		200.	44000.	2000.	0.25	84.1	11.96
			52000.	2000.	0.25	77.3	11.13
			60000.	2000.	0.25	72.0	10.48
			75000.	2000.	0.25	64.4	9.75
		250.	44000.	2000.	0.25	94.0	13.26
			52000.	2000.	0.25	86.5	12.34
			60000.	2000.	0.25	80.5	11.61
			75000.	2000.	0.25	72.0	10.80
		300.	44000.	2000.	0.25	103.0	14.45
			52000.	2000.	0.25	94.7	13.43
			60000.	2000.	0.25	88.2	12.63
			75000.	2000.	0.25	78.9	11.74
		350.	44000.	2000.	0.25	111.2	15.53
			52000.	2000.	0.25	102.3	14.44
			60000.	2000.	0.25	95.2	13.57
			75000.	2000.	0.25	85.2	12.61

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\,D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
28.0	0.7	10.	44000.	6000.	0.47	14.3	3.38
			52000.	2000.	0.25	18.1	3.23
			60000.	2000.	0.25	16.9	3.07
			75000.	2000.	0.25	15.1	2.90
		25.	44000.	6000.	0.47	22.6	4.83
			52000.	2000.	0.25	28.6	4.59
			60000.	2000.	0.25	26.7	4.35
			75000.	2000.	0.25	23.8	4.08
		50.	44000.	6000.	0.47	32.0	6.47
			52000.	2000.	0.25	40.5	6.12
			60000.	2000.	0.25	37.7	5.79
			75000.	2000.	0.25	33.7	5.41
		75.	44000.	6000.	0.47	39.1	7.73
			52000.	2000.	0.25	49.6	7.30
			60000.	2000.	0.25	46.2	6.89
			75000.	2000.	0.25	41.3	6.42
		100.	44000.	6000.	0.47	45.2	8.78
			52000.	2000.	0.25	57.3	8.30
			60000.	2000.	0.25	53.3	7.82
			75000.	2000.	0.25	47.7	7.28
		150.	44000.	6000.	0.47	55.4	10.56
			52000.	2000.	0.25	70.1	9.96
			60000.	2000.	0.25	65.3	9.38
			75000.	2000.	0.25	58.4	8.72
		200.	44000.	6000.	0.47	63.9	12.06
			52000.	2000.	0.25	81.0	11.37
			60000.	2000.	0.25	75.4	10.70
			75000.	2000.	0.25	67.4	9.93
		250.	44000.	6000.	0.47	71.5	13.38
			52000.	2000.	0.25	90.6	12.61
			60000.	2000.	0.25	84.3	11.85
			75000.	2000.	0.25	75.4	11.00
		300.	44000.	6000.	0.47	78.3	14.57
			52000.	2000.	0.25	99.2	13.72
			60000.	2000.	0.25	92.3	12.90
			75000.	2000.	0.25	82.6	11.96
		350.	44000.	6000.	0.47	84.6	15.67
			52000.	2000.	0.25	107.1	14.75
			60000.	2000.	0.25	99.7	13.86
			75000.	2000.	0.25	89.2	12.85

TABLE 3-40 (Cont'd)

MINIMUM IN-PLACE COSTS FOR OVERHEAD FIXED-END, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS WITHOUT WEB REINFORCEMENT

($0.25 \leq \phi_{sc} = e\,\phi_{Lc} \leq 2.00$, $\phi_v = 0$, $2000 \leq f'_c \leq 6000$, $d = 0.9\,D$)

L_S ft	α	q psi	f_{dy} psi	f'_c psi	ϕ_{Sc} psi	D in.	Ct $/sq ft
28.0	0.6	10.	44000.	2000.	0.35	17.4	3.27
			52000.	2000.	0.35	16.0	3.09
			60000.	2000.	0.35	14.9	2.95
			75000.	2000.	0.35	13.3	2.81
		25.	44000.	2000.	0.35	27.5	4.66
			52000.	2000.	0.35	25.3	4.38
			60000.	2000.	0.35	23.5	4.16
			75000.	2000.	0.35	21.0	3.93
		50.	44000.	2000.	0.35	38.8	6.23
			52000.	2000.	0.35	35.7	5.83
			60000.	2000.	0.35	33.3	5.52
			75000.	2000.	0.35	29.7	5.19
		75.	44000.	2000.	0.35	47.6	7.43
			52000.	2000.	0.35	43.8	6.94
			60000.	2000.	0.35	40.7	6.56
			75000.	2000.	0.35	36.4	6.15
		100.	44000.	2000.	0.35	54.9	8.44
			52000.	2000.	0.35	50.5	7.88
			60000.	2000.	0.35	47.0	7.44
			75000.	2000.	0.35	42.1	6.97
		150.	44000.	2000.	0.35	67.3	10.14
			52000.	2000.	0.35	61.9	9.45
			60000.	2000.	0.35	57.6	8.91
			75000.	2000.	0.35	51.5	8.34
		200.	44000.	2000.	0.35	77.7	11.57
			52000.	2000.	0.35	71.5	10.78
			60000.	2000.	0.35	66.5	10.16
			75000.	2000.	0.35	59.5	9.49
		250.	44000.	2000.	0.35	86.8	12.83
			52000.	2000.	0.35	79.9	11.95
			60000.	2000.	0.35	74.4	11.25
			75000.	2000.	0.35	66.5	10.51
		300.	44000.	2000.	0.35	95.1	13.98
			52000.	2000.	0.35	87.5	13.01
			60000.	2000.	0.35	81.5	12.24
			75000.	2000.	0.35	72.9	11.43
		350.	44000.	2000.	0.35	102.8	15.03
			52000.	2000.	0.35	94.5	13.98
			60000.	2000.	0.35	88.0	13.15
			75000.	2000.	0.35	78.7	12.27

3.35 Eccentrically-Loaded Column or Bearing Wall

When the structural members of a reinforced-concrete structure are designed for continuity, eccentric loading of compression members will normally result. For such systems, moment will be transferred to exterior columns from the beams or slabs which the columns support. If the structure is formed from integral roof slabs and bearing walls, the roof slabs will transfer moments to the perimeter bearing walls.

This condition, whereby transverse members transfer shear and moment to column members, is similar to that described in the steel bent analysis of Section 3.24. However, as noted in Section 3.31, reinforced-concrete elements are specifically fabricated for their intended use while structural steel elements, at least insofar as these analyses have assumed, are assembled from standard rolled shapes. Thus, it becomes feasible to design reinforced concrete beams and columns (or, as their design analogue, unit-width strips of roof slab and bearing wall) which will have compatible strength properties for each postulated condition of loading.

It is assumed that lateral earth support will prevent any sidesway of the structure. The beam or slab (roof or floor of structure) resists vertical loads and is rigidly connected to the column or bearing wall. The column or bearing wall supports the vertical reactions of the beam or slab, and also resists lateral loads. The analysis will be developed for a roof slab and bearing wall system, but obviously would be equally applicable to a beam-column system. Thus, a unit length of bearing wall will be designed to support the thrust and moment which are transmitted to it through a rigid connection by a unit width of loaded roof slab.

Depending upon the external conditions of moment and thrust, and upon the internal resistance capacities which are thus developed in the reinforcing steel and concrete, an eccentrically-loaded reinforced bearing wall will deflect about some bending axis. If this neutral axis lies within the column cross-section, both tensile and compressive stresses will develop as a result of eccentric loading. Since it is assumed that concrete cannot resist tension, the tensile flexural strength of a wall element is controlled by the area of tensile steel, the yield strength of the reinforcing steel, and the distance between the neutral axis and the center of gravity

of the tensile steel. Similarly, the compression flexural capacity of the bearing wall is controlled by the effective area and the ultimate compressive strength of the concrete, by the area and yield strength of any reinforcing steel which acts in compression, and by the distance between the neutral axis and the weighted centroid of the compressively-loaded areas.

The location of the neutral axis for a loaded wall section will depend upon the eccentricity and magnitude of the applied loading. However, for a specific wall and a given magnitude of equivalent static load, there may exist some finite eccentricity of load, e_{db}, and a corresponding position of the bending axis such that the wall section is simultaneously loaded to its maximum (P_{db}), both in compression and in tension. (This hypothesis is not valid, however, for cases where the bending axis does not lie within the wall cross-section). If the actual eccentricity, e_d, of the equivalent static load is less than this "balanced" eccentricity, e_{db}, then the ultimate load capacity of the bearing wall will be limited by the compressive stresses which are developed as a result of bending. If $e_d > e_{db}$, the tensile resistance of the wall, and hence the yield capacity of the tensile reinforcement, will control its ultimate load capacity. Since separate equations express the maximum strength of the eccentrically-loaded wall for these two conditions, the initial design requirement is to evaluate e_d and e_{db}.

The ultimate resistance of a short, axially-loaded bearing wall of reinforced concrete, assuming an increase in effective strength of the steel and concrete due to dynamic rates of load application, is expressed in Section 3.32.

$$\frac{P_{do}}{A\, f'_{dc}} = 0.85 + q_{dt} \qquad (3.32.3)$$

where:

P_{do} = the ultimate compressive resistance of a unit length axially loaded bearing wall (lb)

A = the area of a unit length of bearing wall, (sq. in.) For a one-inch length of wall, A is numerically equal to the wall width, D.

f'_{dc} = ultimate compressive strength of dynamically-loaded concrete, related to standard 28-day static cylinder test, (psi)

$$q_{dt} = \frac{A_s f_{dy}}{A\, f'_{dc}}$$

f_{dy} = dynamic yield strength in tension or compression of reinforcing steel, (psi)

A_s = total area of reinforcing steel in a section of the wall taken perpendicular to the line of action for P_{do}, (sq. in.)

By assuming that A_s consists of equal areas of tensile and compressive reinforcement ($A'_s = A''_s = \frac{A_s}{2}$), and by assuming $d_{wall} = 0.9\ D$ wall, Equation 3.32.3 may be written for a unit length of bearing wall as

$$\frac{P_{do}}{d\, f'_{dc}} = 0.945 + 2q_d \qquad (3.35.1)$$

where:

$$q_d = \frac{A'_s\, f_{dy}}{d\, f'_{dc}}$$

A'_s = area of tensile reinforcement

The eccentricity of the balanced equivalent load, referenced to the geometric centroid of the reinforced-concrete bearing wall, can be expressed as

$$e_{db} = \frac{M_{db}}{P_{db}} \qquad (3.35.2)$$

where M_{db}, P_{db} are the applied moment and thrust which, for the particular wall under consideration, simultaneously develop its maximum load capacities in flexure and in compression.

The ultimate equivalent balanced load, P_{db}, can be expressed for a unit width of bearing wall as [37]

$$\frac{P_{db}}{df'_{dc}} = 0.85\, k_1 \left[\frac{90,000}{90,000 + f_{dy}}\right] \qquad (3.35.3)$$

where k_1 is a factor, standard in concrete terminology, which relates the area of compressive concrete to the net area of flexural member.

The ultimate equivalent balanced moment, M_{db}, can be computed from the conditions of equilibrium.[40] By assuming $A'_s = A''_s$ and $d = 0.9\, D$, and by postulating that the compressive steel has strained to its yield stress when the concrete in the compression zone has strained to an arbitrarily-imposed limit of 0.003[40], the following equation can be derived.

$$\frac{M_{db}}{f'_{dc} d^2} = 0.425\, k_1^2 \left[\frac{90,000}{90,000 + f_{dy}}\right]^2 + 0.472\, k_1 \left[\frac{90,000}{90,000 + f_{dy}}\right] + 0.889\, q_d \qquad (3.35.4)$$

Since $e_{db} = \dfrac{M_{db}}{P_{db}}$, assuming $k_1 = 0.85$ for all concrete strengths considered in this study we can write

$$\frac{e_{db}}{d} = 0.555 - \left[\frac{38,250}{90,000 + f_{dy}}\right] + 1.045\, q_d \left[\frac{90,000 + f_{dy}}{76,500}\right] \qquad (3.35.5)$$

It should be noted that e_{db}, as expressed in Equation 3.35.5, is referenced to the geometric centroid of the bearing wall.

When the ultimate bearing capacity of the eccentrically-loaded bearing wall is controlled by the tensile strength of the reinforcing steel, $P_{du} \leq P_{db}$ and $e_d \geq e_{db}$. Incorporating the previously-referenced assumptions, the expression for ultimate column capacity becomes:[37]

$$\frac{P_{du}}{df'_{dc}} = 0.85 \left[\left(0.555 - \frac{e_d}{d}\right) + \sqrt{\left(0.555 - \frac{e_d}{d}\right)^2 + 2.09\, q_d}\right] \qquad (3.35.6)$$

Or, by substituting $\dfrac{M_{du}}{P_{du}} = e_d$, where M_{du} is the ultimate resisting moment of the eccentrically-loaded bearing wall and P_{du} is its ultimate compressive strength Equation 3.35.6 becomes

$$\frac{P_{du}}{df'_{dc}} = 0.85 \left[\left(0.555 - \frac{M_{du}}{dP_{du}}\right) + \sqrt{\left(0.555 - \frac{M_{du}}{P_{du} d}\right)^2 + 2.09\, q_d}\right] \qquad (3.35.7)$$

When the ultimate bearing capacity of the eccentrically-loaded bearing wall is controlled by its compressive strength in flexure, $e_d \leq e_{db}$ and $P_{du} \geq P_{db}$.

The limiting load condition is approached as e_{db} approaches zero, when P_{du} becomes equal to the ultimate axial load P_{do}. For the region $0 < e_d \leq e_{db}$, a linear variation of P_{du} is assumed(40). This yields the equation

$$\frac{P_{du}}{d\, f'_{dc}} = \frac{(0.945 + 2\, q_d)}{1 + \left[\frac{(0.945 + 2\, q_d)\,(90{,}000 + f_{dy})}{65{,}000} - 1\right] \frac{e_d}{e_{db}}} \qquad (3.35.8)$$

Equations 3.35.7 and 3.35.8 express the ultimate bearing wall resistance, $\frac{P_{du}}{d\, f'_{dc}}$, in terms of the material parameters and the ratio $\frac{e_d}{e_{db}}$. Equation 3.35.5 provides a general solution for e_{db}, expressed in terms of the material parameters. The term $e_d = \frac{M_{du}}{P_{du}}$ reflects the relation between the moment and thrust imposed on the wall by the roof slab, and is entirely analogous to the $\frac{M}{P}$ term which appeared in the equations for the steel column bent, Section 3.24. In similar fashion, M_{du} and P_{du} can be related to the loading system for the slab and bearing wall, and thus expressed in general terms.

The axial thrust on the bearing wall, as was assumed for the steel bent, can be taken as the sum of roof slab end-shear and direct load on the column. Thus, for a unit width of slab and wall and an equivalent static load q,

$$P_{du} = V_{slab} + q\, D_{wall} \qquad (3.35.9)$$

Since the wall is assumed to furnish fixed-edge support to the roof slab, the end moment for a unit width of slab with unit equivalent load q is M_{slab}. This moment is taken as the ultimate resisting moment for the slab, since it is assumed that slab design will be based on a controlling flexural mode. In the design of the steel bent, due to the probability of favorable moment readjustments and because of the conservative assumptions as to plastic moment capacity at sections where shear and moment act in combination, the beam end-moment was equated to the axial column moment.

For reinforced concrete analyses, however, since the members are relatively stocky, the increment of column moment due to the eccentricity of shearing load is also included.

By expressing $\frac{M_{du}}{P_{du}}$ ratios for the several types of roof slabs, expressions for ultimate bearing-wall resistance are developed from Equations 3.35.7 and 3.35.8. The lateral loads which the bearing wall must resist, in addition to thrust and moment from the roof or floor slab, are not explicitly included in the analyses. These loads are of appreciably lesser magnitude than the vertical loads, for all shelter designs considered in this study, and their general effect is to increase the capacity of the bearing wall at the wall-slab connection. In rare instances it might be advisable to examine flexural stresses in the central portion of a bearing wall.

(1) Bearing Walls Supporting One-Way Reinforced Slabs

The one-way reinforced slab, when used as a cubicle roof or floor, spans between the side walls of a shelter. For the rigid slab-to-wall connection assumed in this analysis, these side walls are eccentrically loaded by the slab. The end shear transferred to the bearing wall by a unit width of loaded one-way roof slab is $6\ q\ L$, where q is the equivalent static load on the roof slab and L is the clear-span length of unsupported slab. Similarly, still assuming a rigid slab-to-wall connection, the end-moment of a unit width of one-way roof slab is $9\ q\ L^2$.

No reinforcement other than temperature steel is provided in the transverse slab direction, since the slab is considered to resist flexural stresses in one plane only. However, steel detailing must recognize the possible occurrence of localized stresses at the connection between the end wall and the slab. The end wall can be designed either as an axially-loaded bearing wall(Section 3.32)with lateral load, or as a laterally-loaded one-way slab (Section 3.33) which also supports an axial load. The choice between these two approaches is largely one of judgment, and will be influenced by the relative magnitudes of vertical and lateral loading.

The ultimate load capacity of a bearing wall supporting a one-way reinforced slab, assuming a equivalent static load is applied axially with M_{du} approaching zero, can be obtained by substituting $V = 6qL$ in Equation 3.35.9, assuming $d = 0.9D$, and substituting in Equation 3.35.1

$$\frac{q}{f'_{dc}} = \left[\frac{0.1575 + 0.333\, q_d}{\left(\frac{L_{slab}}{d_{wall}}\right) + 0.185}\right] \qquad (3.35.10)$$

The ultimate equivalent load capacity of the bearing wall, assuming a balanced loading ($e_d = e_{db}$, $P_{du} = P_{db}$), is

$$\frac{q}{f'_{dc}} = \left[\frac{10{,}830}{\left(\frac{L_{slab}}{d_{wall}} + 0.185\right)(90{,}000 + f_{dy})}\right] \qquad (3.35.11)$$

The ultimate equivalent load capacity of the bearing wall, assuming a tensile failure in flexure ($e_d \geq e_{db}$, $P_{du} \leq P_{db}$), is

$$\frac{q}{f'_{dc}} = \left[\frac{0.1417}{\frac{L_{slab}}{d_{wall}} + 0.185}\right]\left[\left(0.555 - \frac{M_{du}}{d\,P_{du}}\right) + \sqrt{\left(0.555 - \frac{M_{du}}{d\,P_{du}}\right)^2 + 2.09\, q_d}\,\right] \qquad (3.35.12)$$

where

$$\frac{M_{du}}{P_{du}\, d_{wall}} = 1.5\left(\frac{L_{slab}}{d_{wall}}\right)\left[\frac{1 + 0.370\left(\frac{d_{wall}}{L_{slab}}\right)}{1 + 0.185\left(\frac{d_{wall}}{L_{slab}}\right)}\right] \qquad (3.35.13)$$

The ultimate equivalent load capacity of the bearing wall, assuming a compressive failure in flexure ($e_d \leq e_{db}$, $P_{du} \geq P_{db}$), is

$$\frac{q}{f'_{dc}} = \left[\frac{0.1575 + 0.333\, q_d}{\frac{L_{slab}}{d_{wall}} + 0.185}\right]\left[\frac{1}{1 + \left\{\frac{(0.945 + 2\, q_d)(90{,}000 + f_{dy})}{65{,}000} - 1\right\}\frac{e_d}{e_{db}}}\right] \qquad (3.35.14)$$

Table 3.41 contains computed resistance functions, $\frac{q}{f'_{dc}}$, for bearing walls supporting fixed-end one-way reinforced concrete slabs. This table has been prepared by introducing representative values of f_{dy} and q_d into the equations appearing in this section. The tabulated functions do not provide a unique solution for the width of bearing wall, since various values of d_{wall} can be associated with a given equivalent static load, q, by varying q_d and f_{dy}. Thus, cost analyses must be used to establish the optimum wall depth. The same table can be used to determine design parameters for column members in a rectangular bent, in which case $\frac{12\,B\,q}{f'_{dc}\,b_{column}}$ must be substituted for $\frac{q}{f'_{dc}}$.

(2) Bearing Wall Supporting Two-Way Reinforced Isotropic Slabs

As described for the one-way slab, a two-way roof or floor slab will span between supporting walls. Unlike the one-way slab, however, the two-way slab will transfer shear and moment to all four perimeter walls. For square two-way slabs ($\alpha = 1.0$), moment and shear both remain constant along the entire slab perimeter. For rectangular slabs ($\alpha < 1.0$), the end moments of the slab in the short span (L_S) direction are larger than those in the long span (L_L) direction. Assuming fixed-edge support, the design moments for a two-way reinforced isotropic slab are as follows:

$$M_S = 144\, q\left[\frac{L_S^2}{48}\right]\left[\sqrt{3+\alpha^2} - \alpha\right]^2 \qquad L_S \text{ direction} \qquad (3.35.15)$$

$$M_L = 144\, q\left[\frac{L_S^2}{48}\right]\alpha^2 \qquad L_L \text{ direction}$$

Despite this imbalance of maximum moments, however, the isotropic two-way slab is designed with equal reinforcement in both directions. The average bearing reaction per unit length of bearing wall, assuming the slab is uniformly supported on its four sides, is expressed as,

$$V_{average} = \frac{6\,q\,L_S L_L}{(L_L + L_S)} = \frac{6\,q\,L_S}{(1+\alpha)} \qquad (3.35.16)$$

For the one-way slab, since end walls presumably remain unloaded by the slab,

$$V_{average} = 6\,q\,L \qquad (3.35.16b)$$

Table 3-41

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING

FIXED-EDGE, ONE-WAY REINFORCED CONCRETE SLABS (f_{dy} = 44,000 psi)

q_d	Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft										
Wall	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00
0.020	0.000501	0.001151	0.002104	0.003408	0.005132	0.007367	0.010238	0.013900	0.018542	0.024365	0.031541
0.025	0.000627	0.001437	0.002625	0.004246	0.006379	0.009128	0.012625	0.017026	0.022504	0.029217	0.037271
0.030	0.000752	0.001723	0.003144	0.005079	0.007613	0.010860	0.014953	0.020043	0.026277	0.033771	0.042571
0.035	0.000877	0.002009	0.003661	0.005906	0.008835	0.012564	0.017227	0.022961	0.029886	0.038076	0.047527
0.040	0.001001	0.002293	0.004177	0.006728	0.010044	0.014242	0.019450	0.025789	0.033351	0.042169	0.052197
0.045	0.001126	0.002578	0.004691	0.007545	0.011241	0.015895	0.021625	0.028535	0.036687	0.046078	0.056626
0.050	0.001251	0.002862	0.005203	0.008357	0.012427	0.017524	0.023756	0.031207	0.039909	0.049826	0.060846
0.055	0.001376	0.003145	0.005713	0.009164	0.013601	0.019129	0.025845	0.033809	0.043027	0.053432	0.064890
0.060	0.001500	0.003428	0.006221	0.009966	0.014764	0.020713	0.027894	0.036347	0.046050	0.056910	0.069772
0.065	0.001625	0.003710	0.006728	0.010764	0.015916	0.022275	0.029906	0.038826	0.048987	0.060273	0.072513
0.070	0.001749	0.003992	0.007233	0.011557	0.017058	0.023817	0.031882	0.041249	0.051845	0.063532	0.076126
0.075	0.001873	0.004273	0.007736	0.012345	0.018190	0.025339	0.033825	0.043620	0.054629	0.066697	0.079624
0.080	0.001997	0.004554	0.008237	0.013129	0.019312	0.026843	0.035736	0.045942	0.057346	0.069774	0.083018
0.085	0.002121	0.004835	0.008737	0.013909	0.020424	0.028328	0.037616	0.048218	0.059999	0.072770	0.086315
0.090	0.002245	0.005115	0.009235	0.014684	0.021527	0.029796	0.039467	0.050451	0.062594	0.075693	0.089525
0.095	0.002369	0.005394	0.009731	0.015455	0.022620	0.031246	0.041291	0.052643	0.065133	0.078546	0.092652
0.100	0.002493	0.005673	0.010226	0.016221	0.023705	0.032681	0.043087	0.054797	0.067621	0.081335	0.095705
0.105	0.002617	0.005952	0.010720	0.016984	0.024781	0.034099	0.044859	0.056913	0.070059	0.084063	0.098687
0.110	0.002741	0.006230	0.011211	0.017742	0.025849	0.035502	0.046606	0.058995	0.072452	0.086735	0.101603
0.115	0.002864	0.006507	0.011701	0.018497	0.026908	0.036891	0.048330	0.061044	0.074802	0.089354	0.104458
0.120	0.002988	0.006784	0.012190	0.019247	0.027959	0.038265	0.050031	0.063060	0.077110	0.091923	0.107255
0.125	0.003111	0.007061	0.012677	0.019994	0.029002	0.039625	0.051711	0.065047	0.079379	0.094445	0.109998
0.130	0.003234	0.007337	0.013162	0.020736	0.030037	0.040971	0.053370	0.067004	0.081611	0.096922	0.112689
0.135	0.003358	0.007613	0.013646	0.021475	0.031065	0.042305	0.055009	0.068934	0.083807	0.099356	0.115332
0.140	0.003481	0.007888	0.014128	0.022211	0.032085	0.043625	0.056628	0.070837	0.085970	0.101750	0.117928
0.145	0.003604	0.008163	0.014609	0.022942	0.033098	0.044933	0.058229	0.072715	0.088100	0.104106	0.120481
0.150	0.003727	0.008437	0.015088	0.023670	0.034104	0.046230	0.059812	0.074568	0.090200	0.106424	0.122993
0.155	0.003850	0.008711	0.015566	0.024395	0.035103	0.047514	0.061377	0.076397	0.092270	0.108708	0.125465
0.160	0.003972	0.008984	0.016043	0.025116	0.036096	0.048787	0.062926	0.078204	0.094311	0.110959	0.127899
0.165	0.004095	0.009257	0.016518	0.025834	0.037081	0.050049	0.064458	0.079989	0.096326	0.113177	0.130296
0.170	0.004218	0.009530	0.016991	0.026548	0.038060	0.051301	0.065974	0.081753	0.098314	0.115365	0.132660
0.175	0.004340	0.009802	0.017464	0.027259	0.039033	0.052541	0.067475	0.083497	0.100277	0.117524	0.134990
0.180	0.004463	0.010073	0.017934	0.027966	0.039999	0.053772	0.068961	0.085221	0.102216	0.119654	0.137289
0.185	0.004585	0.010345	0.018404	0.028671	0.040959	0.054992	0.070433	0.086925	0.104132	0.121757	0.139557
0.190	0.004707	0.010615	0.018872	0.029372	0.041913	0.056202	0.071890	0.088612	0.106025	0.123833	0.141795
0.195	0.004830	0.010886	0.019338	0.030070	0.042861	0.057403	0.073334	0.090280	0.107896	0.125885	0.144006
0.200	0.004952	0.011156	0.019804	0.030764	0.043803	0.058595	0.074765	0.091932	0.109747	0.127912	0.146189
0.205	0.005074	0.011425	0.020268	0.031456	0.044740	0.059777	0.076182	0.093566	0.111577	0.129916	0.148346
0.210	0.005196	0.011694	0.020730	0.032145	0.045670	0.060951	0.077588	0.095185	0.113387	0.131897	0.150478
0.215	0.005318	0.011963	0.021191	0.032831	0.046596	0.062116	0.078980	0.096787	0.115178	0.133856	0.152586
0.220	0.005439	0.012231	0.021651	0.033513	0.047516	0.063272	0.080361	0.098374	0.116951	0.135794	0.154670
0.225	0.005561	0.012499	0.022110	0.034193	0.048430	0.064420	0.081731	0.099947	0.118706	0.137711	0.156364
0.230	0.005683	0.012766	0.022567	0.034870	0.049339	0.065560	0.083089	0.101505	0.120443	0.139608	0.157731
0.235	0.005804	0.013033	0.023024	0.035544	0.050244	0.066692	0.084436	0.103048	0.122164	0.141486	0.159097
0.240	0.005926	0.013299	0.023478	0.036215	0.051143	0.067816	0.085772	0.104579	0.123868	0.143346	0.160460
0.245	0.006047	0.013565	0.023932	0.036884	0.052037	0.068932	0.087097	0.106095	0.125557	0.145187	0.161821
0.250	0.006169	0.013831	0.024384	0.037549	0.052926	0.070041	0.088412	0.107599	0.127229	0.147010	0.163180
0.255	0.006290	0.014096	0.024836	0.038212	0.053810	0.071143	0.089718	0.109090	0.128887	0.148237	0.164537
0.260	0.006411	0.014361	0.025285	0.038873	0.054690	0.072237	0.091013	0.110569	0.130530	0.149463	0.165892
0.265	0.006532	0.014626	0.025734	0.039530	0.055565	0.073324	0.092299	0.112035	0.132159	0.150668	0.167246
0.270	0.006653	0.014890	0.026182	0.040186	0.056435	0.074404	0.093575	0.113490	0.133774	0.151910	0.168597
0.275	0.006774	0.015153	0.026628	0.040838	0.057301	0.075478	0.094842	0.114933	0.135376	0.153132	0.169947
0.280	0.006895	0.015417	0.027073	0.041488	0.058163	0.076545	0.096100	0.116366	0.136964	0.154351	0.171296

Table 3-42

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ONE-WAY REINFORCED CONCRETE SLABS (f_{dy} = 52, 000 psi)

q_d	Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft										
Wall	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00
0.020	0.000501	0.001151	0.002104	0.003408	0.005132	0.007367	0.010238	0.013909	0.018542	0.024365	0.031541
0.025	0.000627	0.001437	0.002625	0.004246	0.006379	0.009128	0.012625	0.017026	0.022504	0.029217	0.037271
0.030	0.000752	0.001723	0.003144	0.005079	0.007613	0.010860	0.014953	0.020043	0.026277	0.033771	0.042571
0.035	0.000877	0.002009	0.003661	0.005906	0.008835	0.012564	0.017227	0.022961	0.029886	0.038076	0.047527
0.040	0.001001	0.002293	0.004177	0.006728	0.010044	0.014242	0.019450	0.025789	0.033351	0.042169	0.052197
0.045	0.001126	0.002578	0.004691	0.007545	0.011241	0.015895	0.021625	0.028535	0.036687	0.046078	0.056626
0.050	0.001251	0.002862	0.005203	0.008357	0.012427	0.017524	0.023756	0.031207	0.039909	0.049826	0.060848
0.055	0.001376	0.003145	0.005713	0.009164	0.013601	0.019129	0.025845	0.033809	0.043027	0.053432	0.064890
0.060	0.001500	0.003428	0.006221	0.009966	0.014764	0.020713	0.027894	0.036347	0.046050	0.056910	0.068772
0.065	0.001625	0.003710	0.006728	0.010764	0.015916	0.022275	0.029906	0.038826	0.048987	0.060273	0.072513
0.070	0.001749	0.003992	0.007233	0.011557	0.017058	0.023817	0.031882	0.041249	0.051845	0.063532	0.076126
0.075	0.001873	0.004273	0.007736	0.012345	0.018190	0.025339	0.033825	0.043620	0.054629	0.066697	0.079624
0.080	0.001997	0.004554	0.008237	0.013129	0.019312	0.026843	0.035736	0.045942	0.057346	0.069774	0.083018
0.085	0.002121	0.004835	0.008737	0.013909	0.020424	0.028328	0.037616	0.048218	0.059999	0.072770	0.086315
0.090	0.002245	0.005115	0.009235	0.014684	0.021527	0.029796	0.039467	0.050451	0.062594	0.075693	0.089525
0.095	0.002369	0.005394	0.009731	0.015455	0.022620	0.031246	0.041291	0.052643	0.065133	0.078546	0.092652
0.100	0.002493	0.005673	0.010226	0.016221	0.023705	0.032681	0.043087	0.054797	0.067621	0.081335	0.095705
0.105	0.002617	0.005952	0.010720	0.016984	0.024781	0.034099	0.044859	0.056913	0.070059	0.084063	0.098687
0.110	0.002741	0.006230	0.011211	0.017742	0.025849	0.035502	0.046606	0.058995	0.072452	0.086735	0.101503
0.115	0.002864	0.006507	0.011701	0.018497	0.026908	0.036891	0.048330	0.061044	0.074802	0.089354	0.104458
0.120	0.002988	0.006784	0.012190	0.019247	0.027959	0.038265	0.050031	0.063060	0.077110	0.091923	0.107255
0.125	0.003111	0.007061	0.012677	0.019994	0.029002	0.039625	0.051711	0.065047	0.079379	0.094445	0.109998
0.130	0.003234	0.007337	0.013162	0.020736	0.030037	0.040971	0.053370	0.067004	0.081611	0.096922	0.112689
0.135	0.003358	0.007613	0.013646	0.021475	0.031065	0.042305	0.055009	0.068934	0.083807	0.099356	0.115332
0.140	0.003481	0.007888	0.014128	0.022211	0.032085	0.043625	0.056628	0.070837	0.085970	0.101750	0.117928
0.145	0.003604	0.008163	0.014609	0.022942	0.033098	0.044933	0.058229	0.072715	0.088100	0.104106	0.120481
0.150	0.003727	0.008437	0.015088	0.023670	0.034104	0.046230	0.059812	0.074568	0.090200	0.106424	0.122993
0.155	0.003850	0.008711	0.015566	0.024395	0.035103	0.047514	0.061377	0.076397	0.092270	0.108708	0.125465
0.160	0.003972	0.008984	0.016043	0.025116	0.036096	0.048787	0.062926	0.078204	0.094311	0.110959	0.127899
0.165	0.004095	0.009257	0.016518	0.025834	0.037081	0.050049	0.064458	0.079989	0.096326	0.113177	0.130296
0.170	0.004218	0.009530	0.016991	0.026548	0.038060	0.051301	0.065974	0.081753	0.098314	0.115365	0.132660
0.175	0.004340	0.009802	0.017464	0.027259	0.039033	0.052541	0.067475	0.083497	0.100277	0.117524	0.134990
0.180	0.004463	0.010073	0.017934	0.027966	0.039999	0.053772	0.068961	0.085221	0.102216	0.119654	0.137289
0.185	0.004585	0.010345	0.018404	0.028671	0.040959	0.054992	0.070433	0.086925	0.104132	0.121757	0.139557
0.190	0.004707	0.010615	0.018872	0.029372	0.041913	0.056202	0.071890	0.088612	0.106025	0.123833	0.141795
0.195	0.004830	0.010886	0.019338	0.030070	0.042861	0.057403	0.073334	0.090280	0.107896	0.125885	0.144006
0.200	0.004952	0.011156	0.019804	0.030764	0.043803	0.058595	0.074765	0.091932	0.109747	0.127912	0.146189
0.205	0.005074	0.011425	0.020268	0.031456	0.044740	0.059777	0.076182	0.093566	0.111577	0.129916	0.147832
0.210	0.005196	0.011694	0.020730	0.032145	0.045670	0.060951	0.077588	0.095185	0.113387	0.131897	0.149218
0.215	0.005318	0.011963	0.021191	0.032831	0.046596	0.062116	0.078980	0.096787	0.115178	0.133856	0.150602
0.220	0.005439	0.012231	0.021651	0.033513	0.047516	0.063272	0.080361	0.098374	0.116951	0.135794	0.151984
0.225	0.005561	0.012499	0.022110	0.034193	0.048430	0.064420	0.081731	0.099947	0.118706	0.137711	0.153362
0.230	0.005683	0.012766	0.022567	0.034870	0.049339	0.065560	0.083089	0.101505	0.120443	0.139317	0.154738
0.235	0.005804	0.013033	0.023024	0.035544	0.050244	0.066692	0.084436	0.103048	0.122164	0.140560	0.156112
0.240	0.005926	0.013299	0.023478	0.036215	0.051143	0.067816	0.085772	0.104579	0.123868	0.141801	0.157483
0.245	0.006047	0.013565	0.023932	0.036884	0.052037	0.068932	0.087097	0.106095	0.125557	0.143040	0.158853
0.250	0.006169	0.013831	0.024384	0.037549	0.052926	0.070041	0.088412	0.107599	0.127229	0.144276	0.160220
0.255	0.006290	0.014096	0.024836	0.038212	0.053810	0.071143	0.089718	0.109090	0.128887	0.145511	0.161585
0.260	0.006411	0.014361	0.025285	0.038873	0.054690	0.072237	0.091013	0.110569	0.130391	0.146744	0.162947
0.265	0.006532	0.014626	0.025734	0.039530	0.055565	0.073324	0.092299	0.112035	0.131490	0.147975	0.164308
0.270	0.006653	0.014890	0.026182	0.040186	0.056435	0.074404	0.093575	0.113490	0.132588	0.149205	0.165667
0.275	0.006774	0.015153	0.026628	0.040838	0.057301	0.075478	0.094842	0.114933	0.133684	0.150433	0.167025
0.280	0.006895	0.015417	0.027073	0.041488	0.058163	0.076545	0.096100	0.116366	0.134779	0.151659	0.168380

Table 3-43

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ONE-WAY REINFORCED CONCRETE SLABS (f_{dy} = 60,000 psi)

q_d	Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft										
Wall	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00
0.020	0.000501	0.001151	0.002104	0.003408	0.005132	0.007367	0.010238	0.013900	0.018542	0.024365	0.031541
0.025	0.000627	0.001437	0.002625	0.004246	0.006379	0.009128	0.012625	0.017026	0.022504	0.029217	0.037271
0.030	0.000752	0.001723	0.003144	0.005079	0.007613	0.010860	0.014953	0.020043	0.026277	0.033771	0.042571
0.035	0.000877	0.002009	0.003661	0.005906	0.008835	0.012564	0.017227	0.022961	0.029886	0.038076	0.047527
0.040	0.001001	0.002293	0.004177	0.006728	0.010044	0.014242	0.019450	0.025789	0.033351	0.042169	0.052197
0.045	0.001126	0.002578	0.004691	0.007545	0.011241	0.015895	0.021625	0.028535	0.036687	0.046078	0.056626
0.050	0.001251	0.002862	0.005203	0.008357	0.012427	0.017524	0.023756	0.031207	0.039909	0.049825	0.060848
0.055	0.001376	0.003145	0.005713	0.009164	0.013601	0.019129	0.025845	0.033809	0.043027	0.053432	0.064890
0.060	0.001500	0.003428	0.006221	0.009966	0.014764	0.020713	0.027894	0.036347	0.046050	0.056910	0.068772
0.065	0.001625	0.003710	0.006728	0.010764	0.015916	0.022275	0.029906	0.038826	0.048987	0.060273	0.072513
0.070	0.001749	0.003992	0.007233	0.011557	0.017058	0.023817	0.031882	0.041249	0.051845	0.063532	0.076126
0.075	0.001873	0.004273	0.007736	0.012345	0.018190	0.025339	0.033825	0.043620	0.054629	0.066697	0.079524
0.080	0.001997	0.004554	0.008237	0.013129	0.019312	0.026843	0.035736	0.045942	0.057346	0.069774	0.083018
0.085	0.002121	0.004835	0.008737	0.013909	0.020424	0.028328	0.037616	0.048218	0.059999	0.072770	0.086315
0.090	0.002245	0.005115	0.009235	0.014684	0.021527	0.029796	0.039467	0.050451	0.062594	0.075693	0.089525
0.095	0.002369	0.005394	0.009731	0.015455	0.022620	0.031246	0.041291	0.052643	0.065133	0.078546	0.092652
0.100	0.002493	0.005673	0.010226	0.016221	0.023705	0.032681	0.043087	0.054797	0.067621	0.081335	0.095705
0.105	0.002617	0.005952	0.010720	0.016984	0.024781	0.034099	0.044859	0.056913	0.070059	0.084063	0.098687
0.110	0.002741	0.006230	0.011211	0.017742	0.025849	0.035502	0.046606	0.058995	0.072452	0.086735	0.101603
0.115	0.002864	0.006507	0.011701	0.018497	0.026908	0.036891	0.048330	0.061044	0.074802	0.089354	0.104458
0.120	0.002988	0.006784	0.012190	0.019247	0.027959	0.038265	0.050031	0.063060	0.077110	0.091923	0.107255
0.125	0.003111	0.007061	0.012677	0.019994	0.029002	0.039625	0.051711	0.065047	0.079379	0.094445	0.109998
0.130	0.003234	0.007337	0.013162	0.020736	0.030037	0.040971	0.053370	0.067004	0.081611	0.096922	0.112689
0.135	0.003358	0.007613	0.013646	0.021475	0.031065	0.042305	0.055009	0.068934	0.083807	0.099356	0.115332
0.140	0.003481	0.007888	0.014128	0.022211	0.032085	0.043625	0.056628	0.070837	0.085970	0.101750	0.117928
0.145	0.003604	0.008163	0.014609	0.022942	0.033098	0.044933	0.058229	0.072715	0.088100	0.104106	0.120481
0.150	0.003727	0.008437	0.015088	0.023670	0.034104	0.046230	0.059812	0.074568	0.090200	0.106424	0.122993
0.155	0.003850	0.008711	0.015566	0.024395	0.035103	0.047514	0.061377	0.076397	0.092270	0.108708	0.125465
0.160	0.003972	0.008984	0.016043	0.025116	0.036096	0.048787	0.062926	0.078204	0.094311	0.110959	0.127899
0.165	0.004095	0.009257	0.016518	0.025834	0.037081	0.050049	0.064458	0.079989	0.096326	0.113177	0.130296
0.170	0.004218	0.009530	0.016991	0.026548	0.038060	0.051301	0.065974	0.081753	0.098314	0.115365	0.132660
0.175	0.004340	0.009802	0.017464	0.027259	0.039033	0.052541	0.067475	0.083497	0.100277	0.117524	0.134990
0.180	0.004463	0.010073	0.017934	0.027966	0.039999	0.053772	0.068961	0.085221	0.102216	0.119654	0.137289
0.185	0.004585	0.010345	0.018404	0.028671	0.040959	0.054992	0.070433	0.086925	0.104132	0.121757	0.139380
0.190	0.004707	0.010615	0.018872	0.029372	0.041913	0.056202	0.071890	0.088612	0.106025	0.123833	0.140788
0.195	0.004830	0.010886	0.019338	0.030070	0.042861	0.057403	0.073334	0.090280	0.107896	0.125885	0.142192
0.200	0.004952	0.011156	0.019804	0.030764	0.043803	0.058595	0.074765	0.091932	0.109747	0.127912	0.143592
0.205	0.005074	0.011425	0.020268	0.031456	0.044740	0.059777	0.076182	0.093566	0.111577	0.129916	0.144990
0.210	0.005196	0.011694	0.020730	0.032145	0.045670	0.060951	0.077588	0.095185	0.113387	0.131711	0.146385
0.215	0.005318	0.011963	0.021191	0.032831	0.046596	0.062116	0.078980	0.096787	0.115178	0.132973	0.147776
0.220	0.005439	0.012231	0.021651	0.033513	0.047516	0.063272	0.080361	0.098374	0.116951	0.134227	0.149165
0.225	0.005561	0.012499	0.022110	0.034193	0.048430	0.064420	0.081731	0.099947	0.118706	0.135481	0.150551
0.230	0.005683	0.012766	0.022567	0.034870	0.049339	0.065560	0.083089	0.101505	0.120443	0.136733	0.151935
0.235	0.005804	0.013033	0.023024	0.035544	0.050244	0.066692	0.084436	0.103048	0.122164	0.137982	0.153316
0.240	0.005926	0.013299	0.023478	0.036215	0.051143	0.067816	0.085772	0.104579	0.123635	0.139229	0.154695
0.245	0.006047	0.013565	0.023932	0.036884	0.052037	0.068932	0.087097	0.106095	0.124746	0.140474	0.156071
0.250	0.006169	0.013831	0.024384	0.037549	0.052926	0.070041	0.088412	0.107599	0.125856	0.141717	0.157445
0.255	0.006290	0.014096	0.024836	0.038212	0.053810	0.071143	0.089718	0.109090	0.126964	0.142958	0.158817
0.260	0.006411	0.014361	0.025285	0.038873	0.054690	0.072237	0.091013	0.110569	0.128070	0.144197	0.160186
0.265	0.006532	0.014626	0.025734	0.039530	0.055565	0.073324	0.092299	0.112035	0.129175	0.145434	0.161554
0.270	0.006653	0.014890	0.026182	0.040186	0.056435	0.074404	0.093575	0.113490	0.130277	0.146669	0.162919
0.275	0.006774	0.015153	0.026628	0.040838	0.057301	0.075478	0.094842	0.114780	0.131379	0.147903	0.164283
0.280	0.006895	0.015417	0.027073	0.041488	0.058163	0.076545	0.096100	0.115746	0.132478	0.149134	0.165644

Table 3-44

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ONE-WAY REINFORCED CONCRETE SLABS (f_{dy} = 75,000 psi)

q_d	Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft										
Wall	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50	2.75	3.00
0.020	0.000501	0.001151	0.002104	0.003408	0.005132	0.007367	0.010238	0.013900	0.018542	0.024365	0.031541
0.025	0.000627	0.001437	0.002625	0.004246	0.006379	0.009128	0.012625	0.017026	0.022504	0.029217	0.037271
0.030	0.000752	0.001723	0.003144	0.005079	0.007613	0.010860	0.014953	0.020043	0.026277	0.033771	0.042571
0.035	0.000877	0.002009	0.003661	0.005906	0.008835	0.012564	0.017227	0.022961	0.029886	0.038076	0.047527
0.040	0.001001	0.002293	0.004177	0.006728	0.010044	0.014242	0.019450	0.025789	0.033351	0.042169	0.052197
0.045	0.001126	0.002578	0.004691	0.007545	0.011241	0.015895	0.021625	0.028535	0.036687	0.046078	0.056626
0.050	0.001251	0.002862	0.005203	0.008357	0.012427	0.017524	0.023756	0.031207	0.039909	0.049826	0.060848
0.055	0.001376	0.003145	0.005713	0.009164	0.013601	0.019129	0.025845	0.033809	0.043027	0.053432	0.064890
0.060	0.001500	0.003428	0.006221	0.009966	0.014764	0.020713	0.027894	0.036347	0.046050	0.056910	0.068772
0.065	0.001625	0.003710	0.006728	0.010764	0.015916	0.022275	0.029906	0.038826	0.048987	0.060273	0.072513
0.070	0.001749	0.003992	0.007233	0.011557	0.017058	0.023817	0.031882	0.041249	0.051845	0.063532	0.076126
0.075	0.001873	0.004273	0.007736	0.012345	0.018190	0.025339	0.033825	0.043620	0.054629	0.066697	0.079624
0.080	0.001997	0.004554	0.008237	0.013129	0.019312	0.026843	0.035736	0.045942	0.057346	0.069774	0.083018
0.085	0.002121	0.004835	0.008737	0.013909	0.020424	0.028328	0.037616	0.048218	0.059999	0.072770	0.086315
0.090	0.002245	0.005115	0.009235	0.014684	0.021527	0.029796	0.039467	0.050451	0.062594	0.075693	0.089525
0.095	0.002369	0.005394	0.009731	0.015455	0.022620	0.031246	0.041291	0.052643	0.065133	0.078546	0.092652
0.100	0.002493	0.005673	0.010226	0.016221	0.023705	0.032681	0.043087	0.054797	0.067621	0.081335	0.095705
0.105	0.002617	0.005952	0.010720	0.016984	0.024781	0.034099	0.044859	0.056913	0.070059	0.084063	0.098587
0.110	0.002741	0.006230	0.011211	0.017742	0.025849	0.035502	0.046606	0.058995	0.072452	0.086735	0.101503
0.115	0.002864	0.006507	0.011701	0.018497	0.026908	0.036891	0.048330	0.061044	0.074802	0.089354	0.104458
0.120	0.002988	0.006784	0.012190	0.019247	0.027959	0.038265	0.050031	0.063060	0.077110	0.091923	0.107255
0.125	0.003111	0.007061	0.012677	0.019994	0.029002	0.039625	0.051711	0.065047	0.079379	0.094445	0.109998
0.130	0.003234	0.007337	0.013162	0.020736	0.030037	0.040971	0.053370	0.067004	0.081611	0.096922	0.112689
0.135	0.003358	0.007613	0.013646	0.021475	0.031065	0.042305	0.055009	0.068934	0.083807	0.099356	0.115332
0.140	0.003481	0.007888	0.014128	0.022211	0.032085	0.043625	0.056628	0.070837	0.085970	0.101750	0.117928
0.145	0.003604	0.008163	0.014609	0.022942	0.033098	0.044933	0.058229	0.072715	0.088100	0.104106	0.120481
0.150	0.003727	0.008437	0.015088	0.023670	0.034104	0.046230	0.059812	0.074568	0.090200	0.106424	0.122993
0.155	0.003850	0.008711	0.015566	0.024395	0.035103	0.047514	0.061377	0.076397	0.092270	0.108708	0.125465
0.160	0.003972	0.008984	0.016043	0.025116	0.036096	0.048787	0.062926	0.078204	0.094311	0.110959	0.127323
0.165	0.004095	0.009257	0.016518	0.025834	0.037081	0.050049	0.064458	0.079939	0.096326	0.113177	0.128764
0.170	0.004218	0.009530	0.016991	0.026548	0.038060	0.051301	0.065974	0.081753	0.098314	0.115365	0.130200
0.175	0.004340	0.009802	0.017464	0.027259	0.039033	0.052541	0.067475	0.083497	0.100277	0.117524	0.131633
0.180	0.004463	0.010073	0.017934	0.027966	0.039999	0.053772	0.068961	0.085221	0.102216	0.119654	0.133061
0.185	0.004585	0.010345	0.018404	0.028671	0.040959	0.054992	0.070433	0.086925	0.104132	0.120871	0.134485
0.190	0.004707	0.010615	0.018872	0.029372	0.041913	0.056202	0.071890	0.088612	0.106025	0.122156	0.135906
0.195	0.004830	0.010886	0.019338	0.030070	0.042861	0.057403	0.073334	0.090280	0.107896	0.123438	0.137323
0.200	0.004952	0.011156	0.019804	0.030764	0.043803	0.058595	0.074765	0.091932	0.109747	0.124717	0.138737
0.205	0.005074	0.011425	0.020268	0.031456	0.044740	0.059777	0.076182	0.093566	0.111577	0.125992	0.140147
0.210	0.005196	0.011694	0.020730	0.032145	0.045670	0.060951	0.077588	0.095185	0.112881	0.127265	0.141554
0.215	0.005318	0.011963	0.021191	0.032831	0.046596	0.062116	0.078980	0.096787	0.114014	0.128535	0.142958
0.220	0.005439	0.012231	0.021651	0.033513	0.047516	0.063272	0.080361	0.098374	0.115145	0.129802	0.144359
0.225	0.005561	0.012499	0.022110	0.034193	0.048430	0.064420	0.081731	0.099947	0.116274	0.131067	0.145758
0.230	0.005683	0.012766	0.022567	0.034870	0.049339	0.065560	0.083089	0.101505	0.117401	0.132329	0.147153
0.235	0.005804	0.013033	0.023024	0.035544	0.050244	0.066692	0.084436	0.103048	0.118525	0.133588	0.148545
0.240	0.005926	0.013299	0.023478	0.036215	0.051143	0.067816	0.085772	0.104406	0.119647	0.134846	0.149935
0.245	0.006047	0.013565	0.023932	0.036884	0.052037	0.068932	0.087097	0.105390	0.120767	0.136100	0.151323
0.250	0.006169	0.013831	0.024384	0.037549	0.052926	0.070041	0.088412	0.106372	0.121885	0.137353	0.152708
0.255	0.006290	0.014096	0.024836	0.038212	0.053810	0.071143	0.089718	0.107352	0.123002	0.138603	0.154090
0.260	0.006411	0.014361	0.025285	0.038873	0.054690	0.072237	0.091013	0.108331	0.124116	0.139852	0.155470
0.265	0.006532	0.014626	0.025734	0.039530	0.055565	0.073324	0.092299	0.109308	0.125228	0.141098	0.156848
0.270	0.006653	0.014890	0.026182	0.040186	0.056435	0.074404	0.093575	0.110283	0.126339	0.142342	0.158224
0.275	0.006774	0.015153	0.026628	0.040838	0.057301	0.075478	0.094842	0.111257	0.127448	0.143585	0.159598
0.280	0.006895	0.015417	0.027073	0.041488	0.058163	0.076545	0.095941	0.112229	0.128555	0.144825	0.160969

Thus, for a square slab ($\alpha = 1.0$) the average edge bearing due to a one-way slab will be one-half that due to a two-way slab. For very long slabs ($\alpha \to 0$), the edge bearing will be approximately the same for both types of slabs.

In fact, however, the edge bearing is not uniform along the supported edges of a slab[39]. This non-uniformity, which was implicitly ignored in the analysis of bearing walls supporting one-way slabs, may be of greater concern when two-way slabs are considered. Other studies[2] have recognized the effects of two-way load distribution by proposing that the ultimate shearing capacity of a one-way slab ($V = 0.22\, d\, f'_{dc}$) be increased according to the following relationship.

$$V_{two\text{-}way} = V_{one\text{-}way} \, \frac{2}{3} (1 + \alpha) \quad \text{for } \alpha \geq 0.5 \qquad (3.35.17)$$

By implication, the maximum slab reaction on the bearing wall is thus considered as $6\, q\, L_S \times \dfrac{3}{2(1+\alpha)} = \dfrac{9\, q\, L_S}{(1+\alpha)}$. The relation between maximum end shear and average end shear for an isotropic two-way slab then becomes,

$$\frac{V_{maximum}}{V_{average}} = \frac{\left[\dfrac{9\, q\, L_S}{(1+\alpha)}\right]}{\left[\dfrac{6\, q\, L_S}{(1+\alpha)}\right]} = 1.5 \qquad (3.35.18)$$

In order to obtain general design expressions for bearing walls supporting loaded slabs, the ratio of moment to shear at the slab edge is required (Equations 3.35.7 and 3.35.8). There is some uncertainty as to the most realistic M/V ratio for slabs in general, and for two-way slabs in particular. As an approximation, reasoning that the end walls should be able to support the ultimate capacity of the slab reinforcement, the end bearing walls supporting an isotropic two-way slab will be designed for the same loading conditions as are assumed for side walls. Thus, for a fixed-end isotropic slab and equivalent static load q ,

$$
\left.
\begin{aligned}
V_{slab} &= \frac{6\,q\,L_S}{(1+\alpha)} \\
P_{wall} &= \frac{6\,q\,L_S}{(1+\alpha)} + q\,D_{wall} \\
M_{slab} &= 3q\,L_S^{\,2}\left[\sqrt{3+\alpha^2}-\alpha\right]^2 \\
M_{wall} &= 3q\,L_S^{\,2}\left[\sqrt{3+\alpha^2}-\alpha\right]^2 + \frac{3\,q\,L_S\,D_{wall}}{(1+\alpha)}
\end{aligned}
\right\} \qquad (3.35.19)
$$

The ultimate axial load capacity of the bearing wall, assuming equivalent static loading, is

$$
\frac{q}{f'_{dc}} = \left[\frac{0.1575 + 0.333\,q_d}{\dfrac{L_S}{(1+\alpha)\,d} + 0.185}\right] \qquad (3.35.20)
$$

where

$\alpha = \dfrac{L_S}{L_L}$

L_S = clear length of slab in short span direction, (ft)

L_L = clear length of slab in long span direction (ft)

d = cross-sectional depth of bearing wall, (in.)

For balanced equivalent loading ($e_d = e_{db}$, $P_{du} = P_{db}$) the ultimate capacity of the bearing wall is ,

$$
\frac{q}{f'_{dc}} = \frac{10{,}830}{\left[\dfrac{L_S}{(1+\alpha)\,d} + 0.185\right]\left[90{,}000 + f_{dy}\right]} \qquad (3.35.21)
$$

Assuming a tensile failure due to flexural loading ($e_d \geq e_{db}$, $P_{du} \leq P_{db}$) , the ultimate equivalent load capacity of the bearing wall is,

$$\frac{q}{f'_{dc}} = \left[\frac{0.1417}{\frac{L_S}{(1+\alpha)\,d} + 0.185}\right]\left[\left(0.555 - \frac{M_{du}}{d\,P_{du}}\right) + \sqrt{\left(0.555 - \frac{M_{du}}{d\,P_{du}}\right)^2 + 2.09\,q_d}\,\right] \tag{3.35.22}$$

where

$$\frac{M_{du}}{P_{du}\,d_{wall}} = 1.5\,L_S\left[\frac{\frac{\left(\sqrt{3+\alpha^2}-\alpha\right)^2}{3} + \frac{0.370\,d/L_S}{(1+\alpha)}}{\frac{1}{(1+\alpha)} + 0.185\,d/L_S}\right] \tag{3.35.23}$$

The ultimate equivalent load capacity of the bearing wall, assuming a compressive failure in flexure ($e_d \leq e_{db}$, $P_{du} \geq P_{db}$), is

$$\frac{q}{f'_{dc}} = \left[\frac{0.1575 + 0.333\,q_d}{\frac{L_S}{(1+\alpha)\,d_{wall}} + 0.185}\right]\left[\frac{1}{1 + \left\{\frac{(0.945 + 2\,q_d)(90{,}000 + f_{dy})}{65{,}000} - 1\right\}\frac{e_d}{e_{db}}}\right] \tag{3.35.24}$$

Table 3.45 contains computed resistance functions, $\frac{q}{f'_{dc}}$, for bearing walls supporting two-way isotropic reinforced concrete slabs with fixed ends. This table is used in similar fashion to that described for one-way slabs (Tables 3.41 to 3.44 inclusive).

(3) Bearing Walls Supporting Two-way Reinforced Orthotropic Slabs

For $\alpha = 1.0$, designs are identical for orthotropic and isotropic two-way slabs. For values of $\alpha < 1.0$, however, the flexural reinforcement in the long span direction is less for the orthotropic slab than for its isotropic counterpart. Side bearing walls for orthotropic slabs are analyzed as described for walls supporting isotropic slabs. The behavior of end walls supporting orthotropic two-way slabs will be intermediate between that of end walls supporting one-way slabs, where no slab-to-wall moment transfer is assumed, and the functioning of end walls in combination with isotropic two-way slabs. Separate design analyses, accordingly, will be supplied for end walls and side walls when loading is applied through orthotropically-reinforced slabs.

The edge moment and shear for the orthotropic slab are computed as described for the isotropic slab, after the introduction[39] of the

Table 3-45

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS (f_{dy} = 44,000 psi)

q_d Wall	α	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
		Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft							
0.03	0.7	0.0039543	0.0074615	0.0125818	0.0198416	0.0298670	0.0431815	0.0598677	0.0792740
0.03	0.8	0.0044345	0.0084242	0.0143246	0.0227942	0.0345676	0.0501234	0.0692597	0.0910461
0.03	0.9	0.0049652	0.0095011	0.0162994	0.0261710	0.0399362	0.0579233	0.0795268	0.1034369
0.03	1.0	0.0055500	0.0107031	0.0185324	0.0300151	0.0460059	0.0665447	0.0905356	0.1163519
0.04	0.7	0.0052523	0.0098647	0.0164982	0.0256700	0.0378700	0.0533193	0.0717462	0.0924125
0.04	0.8	0.0058869	0.0111222	0.0187277	0.0293231	0.0434241	0.0611078	0.0818095	0.1045140
0.04	0.9	0.0065871	0.0125235	0.0212343	0.0334429	0.0496419	0.0696691	0.0926072	0.1171936
0.04	1.0	0.0073574	0.0140811	0.0240425	0.0380591	0.0565262	0.0789397	0.1040062	0.1302794
0.05	0.7	0.0065406	0.0122293	0.0202970	0.0312072	0.0452813	0.0624772	0.0822908	0.1039039
0.05	0.8	0.0073269	0.0137702	0.0229779	0.0354775	0.0515459	0.0709462	0.0928972	0.1163818
0.05	0.9	0.0081932	0.0154816	0.0259712	0.0402398	0.0584567	0.0801123	0.1041357	0.1293373
0.05	1.0	0.0091448	0.0173760	0.0292983	0.0455087	0.0659951	0.0899034	0.1158758	0.1426135
0.06	0.7	0.0078195	0.0145571	0.0239883	0.0364930	0.0522155	0.0708940	0.0918701	0.1142953
0.06	0.8	0.0087549	0.0163710	0.0270901	0.0413151	0.0590902	0.0799313	0.1029390	0.1271123
0.06	0.9	0.0097861	0.0183791	0.0305324	0.0466421	0.0665885	0.0896090	0.1145608	0.1403296
0.06	1.0	0.0109128	0.0205937	0.0343321	0.0524789	0.0746762	0.0998403	0.1266076	0.1538012
0.07	0.7	0.0090890	0.0168497	0.0275806	0.0415588	0.0587544	0.0787251	0.1007087	0.1238528
0.07	0.8	0.0101711	0.0189270	0.0310770	0.0468805	0.0661651	0.0882573	0.1121848	0.1369808
0.07	0.9	0.0113601	0.0212196	0.0349359	0.0527133	0.0741751	0.0983777	0.1241493	0.1504468
0.07	1.0	0.0126620	0.0237394	0.0391699	0.0590519	0.0827387	0.1089943	0.1364772	0.1641128
0.08	0.7	0.0103496	0.0191088	0.0310816	0.0464299	0.0649587	0.0860778	0.1089558	0.1327497
0.08	0.8	0.0115758	0.0214404	0.0349493	0.0522085	0.0728486	0.0960496	0.1207984	0.1461668
0.08	0.9	0.0129216	0.0240065	0.0391971	0.0584995	0.0813133	0.1065640	0.1330751	0.1598696
0.08	1.0	0.0143929	0.0268177	0.0438329	0.0652886	0.0902983	0.1175254	0.1456640	0.1737267
0.09	0.7	0.0116012	0.0213357	0.0344979	0.0511272	0.0708750	0.0930306	0.1167163	0.1411067
0.09	0.8	0.0129694	0.0239134	0.0387164	0.0573271	0.0791992	0.1033994	0.1288939	0.1547968
0.09	0.9	0.0144690	0.0267425	0.0433290	0.0640374	0.0880743	0.1142704	0.1414593	0.1687239
0.09	1.0	0.0161062	0.0298328	0.0483388	0.0712361	0.0974390	0.1255459	0.1542928	0.1827679
0.10	0.7	0.0128441	0.0235318	0.0378353	0.0556681	0.0765402	0.0996422	0.1240674	0.1490114
0.10	0.8	0.0143521	0.0263477	0.0423865	0.0622592	0.0852623	0.1103744	0.1365551	0.1629557
0.10	0.9	0.0160028	0.0294305	0.0473426	0.0693565	0.0945123	0.1215725	0.1493898	0.1771017
0.10	1.0	0.0178024	0.0327884	0.0527025	0.0769310	0.1042235	0.1331377	0.1624543	0.1913279
0.11	0.7	0.0140786	0.0256983	0.0410990	0.0600673	0.0819836	0.1059586	0.1310677	0.1565303
0.11	0.8	0.0157240	0.0287452	0.0459665	0.0670238	0.0910737	0.1170267	0.1438451	0.1707180
0.11	0.9	0.0175231	0.0320729	0.0512477	0.0744808	0.1006696	0.1285281	0.1569331	0.1850725
0.11	1.0	0.0194820	0.0356879	0.0569366	0.0824032	0.1107004	0.1403631	0.1702171	0.1994763
0.12	0.7	0.0153047	0.0278364	0.0442938	0.0643372	0.0872295	0.1120159	0.1377632	0.1637148
0.12	0.8	0.0170856	0.0311074	0.0494630	0.0716370	0.0966623	0.1233973	0.1508131	0.1781349
0.12	0.9	0.0190305	0.0346718	0.0550524	0.0794302	0.1065800	0.1351822	0.1641411	0.1926904
0.12	1.0	0.0211454	0.0385343	0.0610521	0.0876768	0.1169079	0.1472704	0.1776345	0.2072673

Table 3-46

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS (f_{dy} = 52, 000 psi)

q_d		Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft							
Wall	α	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
0.03	0.7	0.0039543	0.0074615	0.0125818	0.0198416	0.0298670	0.0431815	0.0598677	0.0793740
0.03	0.8	0.0044345	0.0084242	0.0143246	0.0227942	0.0345676	0.0501234	0.0692597	0.0910461
0.03	0.9	0.0049652	0.0095011	0.0162994	0.0261710	0.0399362	0.0579233	0.0795268	0.1034369
0.03	1.0	0.0055500	0.0107031	0.0185324	0.0300151	0.0460059	0.0665447	0.0905356	0.1163519
0.04	0.7	0.0052523	0.0098647	0.0164982	0.0256700	0.0378700	0.0533193	0.0717462	0.0924125
0.04	0.8	0.0058869	0.0111222	0.0187277	0.0293231	0.0434241	0.0611078	0.0818095	0.1045140
0.04	0.9	0.0065871	0.0125235	0.0212343	0.0334429	0.0496419	0.0696691	0.0926072	0.1171936
0.04	1.0	0.0073574	0.0140811	0.0240425	0.0380591	0.0565262	0.0789397	0.1040062	0.1302794
0.05	0.7	0.0065406	0.0122293	0.0202970	0.0312072	0.0452813	0.0624772	0.0822908	0.1039039
0.05	0.8	0.0073269	0.0137702	0.0229779	0.0354775	0.0515459	0.0709442	0.0928972	0.1163818
0.05	0.9	0.0081932	0.0154816	0.0259712	0.0402390	0.0584567	0.0801123	0.1041357	0.1293373
0.05	1.0	0.0091448	0.0173760	0.0292983	0.0455087	0.0659951	0.0899034	0.1158758	0.1426135
0.06	0.7	0.0078195	0.0145571	0.0239883	0.0364930	0.0522155	0.0708940	0.0918701	0.1142953
0.06	0.8	0.0087549	0.0163710	0.0270901	0.0413151	0.0590902	0.0799313	0.1029390	0.1271123
0.06	0.9	0.0097341	0.0183791	0.0305324	0.0466621	0.0665885	0.0896090	0.1145608	0.1403296
0.06	1.0	0.0109128	0.0205937	0.0343321	0.0524789	0.0746762	0.0998403	0.1266076	0.1538012
0.07	0.7	0.0090890	0.0168497	0.0275806	0.0415588	0.0587544	0.0787251	0.1007087	0.1238528
0.07	0.8	0.0101711	0.0189270	0.0310770	0.0468805	0.0661651	0.0882573	0.1121848	0.1369808
0.07	0.9	0.0113601	0.0212196	0.0349359	0.0527133	0.0741751	0.0983777	0.1241493	0.1504468
0.07	1.0	0.0126620	0.0237394	0.0391699	0.0590519	0.0827387	0.1089943	0.1364772	0.1641128
0.08	0.7	0.0103496	0.0191088	0.0310816	0.0464299	0.0649587	0.0860778	0.1089558	0.1327497
0.08	0.8	0.0115758	0.0214404	0.0349493	0.0522085	0.0728486	0.0960496	0.1207984	0.1461668
0.08	0.9	0.0129216	0.0240065	0.0391971	0.0584995	0.0813133	0.1065640	0.1330751	0.1598696
0.08	1.0	0.0143929	0.0268177	0.0438329	0.0652886	0.0902983	0.1175254	0.1456640	0.1737267
0.09	0.7	0.0116012	0.0213357	0.0344979	0.0511272	0.0708750	0.0930306	0.1167163	0.1411067
0.09	0.8	0.0129694	0.0239134	0.0387164	0.0573271	0.0791992	0.1033994	0.1288939	0.1547948
0.09	0.9	0.0144690	0.0267425	0.0433290	0.0640374	0.0880743	0.1142704	0.1414593	0.1687239
0.09	1.0	0.0161062	0.0298328	0.0483388	0.0712361	0.0974390	0.1255459	0.1542928	0.1827679
0.10	0.7	0.0128441	0.0235318	0.0378353	0.0556681	0.0765402	0.0996422	0.1240674	0.1490114
0.10	0.8	0.0143521	0.0263477	0.0423865	0.0622592	0.0852623	0.1103744	0.1365551	0.1629557
0.10	0.9	0.0160028	0.0294305	0.0473426	0.0693565	0.0945123	0.1215725	0.1493898	0.1771017
0.10	1.0	0.0178024	0.0327884	0.0527025	0.0769310	0.1042235	0.1331377	0.1624543	0.1913279
0.11	0.7	0.0140786	0.0256983	0.0410990	0.0600673	0.0819836	0.1059586	0.1310677	0.1565303
0.11	0.8	0.0157240	0.0287452	0.0459665	0.0670238	0.0910737	0.1170267	0.1438451	0.1707180
0.11	0.9	0.0175231	0.0320729	0.0512477	0.0744808	0.1006696	0.1285281	0.1569331	0.1850725
0.11	1.0	0.0194820	0.0356879	0.0569366	0.0824032	0.1107004	0.1403631	0.1702171	0.1988230
0.12	0.7	0.0153047	0.0278364	0.0442938	0.0643372	0.0872295	0.1120159	0.1377632	0.1637148
0.12	0.8	0.0170656	0.0311074	0.0494630	0.0716370	0.0966623	0.1233973	0.1508131	0.1781349
0.12	0.9	0.0190305	0.0346718	0.0550524	0.0794302	0.1065800	0.1351822	0.1641411	0.1925904
0.12	1.0	0.0211454	0.0385343	0.0610521	0.0875768	0.1169079	0.1472704	0.1776345	0.2034903

Table 3-47

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS (f_{dy} = 60, 000 psi)

q_d Wall	α	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
		Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft							
0.03	0.7	0.0039543	0.0074615	0.0125818	0.0198416	0.0298670	0.0431815	0.0598677	0.0793740
0.03	0.8	0.0044345	0.0084242	0.0143246	0.0227942	0.0345676	0.0501234	0.0692597	0.0910461
0.03	0.9	0.0049652	0.0095011	0.0162994	0.0261710	0.0399362	0.0579233	0.0795268	0.1034369
0.03	1.0	0.0055500	0.0107031	0.0185324	0.0300151	0.0460059	0.0665447	0.0905356	0.1163519
0.04	0.7	0.0052523	0.0098647	0.0164982	0.0256700	0.0378700	0.0533193	0.0717462	0.0924125
0.04	0.8	0.0058869	0.0111222	0.0187277	0.0293231	0.0434241	0.0611078	0.0818095	0.1045140
0.04	0.9	0.0065871	0.0125235	0.0212343	0.0334429	0.0496419	0.0696691	0.0926072	0.1171936
0.04	1.0	0.0073574	0.0140811	0.0240425	0.0380591	0.0565262	0.0789397	0.1040062	0.1302794
0.05	0.7	0.0065406	0.0122293	0.0202970	0.0312072	0.0452813	0.0624772	0.0822908	0.1039039
0.05	0.8	0.0073269	0.0137702	0.0229779	0.0354775	0.0515459	0.0709442	0.0928972	0.1163818
0.05	0.9	0.0081932	0.0154816	0.0259712	0.0402390	0.0584567	0.0801123	0.1041357	0.1293373
0.05	1.0	0.0091448	0.0173760	0.0292983	0.0455087	0.0659951	0.0899034	0.1158758	0.1426135
0.06	0.7	0.0078195	0.0145571	0.0239883	0.0364930	0.0522155	0.0708940	0.0918701	0.1142953
0.06	0.8	0.0087549	0.0163710	0.0270901	0.0413151	0.0590902	0.0799313	0.1029390	0.1271123
0.06	0.9	0.0097841	0.0183791	0.0305324	0.0466421	0.0665885	0.0896090	0.1145608	0.1403296
0.06	1.0	0.0109128	0.0205937	0.0343321	0.0524789	0.0746762	0.0998403	0.1266076	0.1538012
0.07	0.7	0.0090890	0.0168497	0.0275806	0.0415588	0.0587544	0.0787251	0.1007087	0.1238[illegible]28
0.07	0.8	0.0101711	0.0189270	0.0310770	0.0468805	0.0661651	0.0882573	0.1121848	0.1369809
0.07	0.9	0.0113601	0.0212196	0.0349359	0.0527133	0.0741751	0.0983777	0.1241493	0.1504468
0.07	1.0	0.0126620	0.0237394	0.0391699	0.0590519	0.0827387	0.1089943	0.1364772	0.1641128
0.08	0.7	0.0103496	0.0191088	0.0310816	0.0464299	0.0649587	0.0860778	0.1089558	0.1327497
0.08	0.8	0.0115758	0.0214404	0.0349493	0.0522085	0.0728486	0.0960496	0.1207984	0.1461668
0.08	0.9	0.0129216	0.0240065	0.0391971	0.0584995	0.0813133	0.1065640	0.1330751	0.1598696
0.08	1.0	0.0143929	0.0268177	0.0438329	0.0652886	0.0902983	0.1175254	0.1456640	0.1737267
0.09	0.7	0.0116012	0.0213357	0.0344979	0.0511272	0.0708750	0.0930306	0.1167163	0.1411067
0.09	0.8	0.0129694	0.0239134	0.0387164	0.0573271	0.0791992	0.1033994	0.1288939	0.1547948
0.09	0.9	0.0144690	0.0267425	0.0433290	0.0640374	0.0880743	0.1142704	0.1414593	0.1687239
0.09	1.0	0.0161062	0.0298328	0.0483388	0.0712361	0.0974390	0.1255459	0.1542928	0.1827579
0.10	0.7	0.0128441	0.0235318	0.0378353	0.0556681	0.0765402	0.0996422	0.1240674	0.1490114
0.10	0.8	0.0143521	0.0263477	0.0423865	0.0622592	0.0852623	0.1103744	0.1365551	0.1629557
0.10	0.9	0.0160028	0.0294305	0.0473426	0.0693565	0.0945123	0.1215725	0.1493898	0.1771017
0.10	1.0	0.0178324	0.0327884	0.0527025	0.0769310	0.1042235	0.1331377	0.1624543	0.1896110
0.11	0.7	0.0140786	0.0256983	0.0410990	0.0600673	0.0819836	0.1059586	0.1310677	0.1565303
0.11	0.8	0.0157240	0.0287452	0.0459665	0.0670238	0.0910737	0.1170267	0.1438451	0.1707180
0.11	0.9	0.0175231	0.0320729	0.0512477	0.0744908	0.1006696	0.1285281	0.1569331	0.1839202
0.11	1.0	0.0194920	0.0356879	0.0569366	0.0824032	0.1107004	0.1403631	0.1702171	0.1943594
0.12	0.7	0.0153047	0.0278364	0.0442938	0.0643372	0.0872295	0.1120159	0.1377632	0.1637148
0.12	0.8	0.0170856	0.0311074	0.0494630	0.0716370	0.0966623	0.1233973	0.1508131	0.1777081
0.12	0.9	0.0190505	0.0346718	0.0550524	0.0794302	0.1065800	0.1351822	0.1641411	0.1883976
0.12	1.0	0.0211454	0.0385343	0.0610521	0.0876768	0.1169079	0.1472704	0.1774170	0.1990568

Table 3-48

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ISOTROPIC TWO-WAY REINFORCED CONCRETE SLABS (f_{dy} = 75, 000 psi)

q_d Wall	α	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
		Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft							
0.03	0.7	0.0039543	0.0074615	0.0125818	0.0198416	0.0298670	0.0431815	0.0598677	0.0793740
0.03	0.8	0.0044345	0.0084242	0.0143246	0.0227942	0.0345676	0.0501234	0.0692597	0.0910461
0.03	0.9	0.0049652	0.0095011	0.0162994	0.0261710	0.0399362	0.0579233	0.0795268	0.1034369
0.03	1.0	0.0055500	0.0107031	0.0185324	0.0300151	0.0460059	0.0665447	0.0905356	0.1163519
0.04	0.7	0.0052523	0.0098647	0.0164982	0.0256700	0.0378700	0.0533193	0.0717462	0.0924125
0.04	0.8	0.0058869	0.0111222	0.0187277	0.0293231	0.0434241	0.0611078	0.0818095	0.1045140
0.04	0.9	0.0065871	0.0125235	0.0212343	0.0334429	0.0496419	0.0696691	0.0926072	0.1171936
0.04	1.0	0.0073574	0.0140811	0.0240425	0.0380591	0.0565262	0.0789397	0.1040062	0.1302794
0.05	0.7	0.0065406	0.0122293	0.0202970	0.0312072	0.0452813	0.0624772	0.0822908	0.1039039
0.05	0.8	0.0073269	0.0137702	0.0229779	0.0354775	0.0515459	0.0709442	0.0928972	0.1163818
0.05	0.9	0.0081932	0.0154816	0.0259712	0.0402390	0.0584567	0.0801123	0.1041357	0.1293373
0.05	1.0	0.0091448	0.0173760	0.0292983	0.0455087	0.0659951	0.0899034	0.1158758	0.1426135
0.06	0.7	0.0078195	0.0145571	0.0239883	0.0364930	0.0522155	0.0708940	0.0918701	0.1142953
0.06	0.8	0.0087549	0.0163710	0.0270901	0.0413151	0.0590902	0.0799313	0.1029390	0.1271123
0.06	0.9	0.0097841	0.0183791	0.0305324	0.0466421	0.0665885	0.0896090	0.1145608	0.1403296
0.06	1.0	0.0109128	0.0205937	0.0343321	0.0524789	0.0746762	0.0998403	0.1266076	0.1538012
0.07	0.7	0.0090890	0.0168497	0.0275806	0.0415588	0.0587544	0.0787251	0.1007087	0.1238528
0.07	0.8	0.0101711	0.0189270	0.0310770	0.0468805	0.0661651	0.0882573	0.1121848	0.1369808
0.07	0.9	0.0113601	0.0212196	0.0349359	0.0527133	0.0741751	0.0983777	0.1241493	0.1504468
0.07	1.0	0.0126620	0.0237394	0.0391699	0.0590519	0.0827387	0.1089943	0.1364772	0.1641128
0.08	0.7	0.0103496	0.0191088	0.0310816	0.0464299	0.0649587	0.0860778	0.1089558	0.1327497
0.08	0.8	0.0115758	0.0214404	0.0349493	0.0522085	0.0728486	0.0960496	0.1207984	0.1461568
0.08	0.9	0.0129216	0.0240065	0.0391971	0.0584995	0.0813133	0.1065640	0.1330751	0.1598696
0.08	1.0	0.0143929	0.0268177	0.0438329	0.0652886	0.0902983	0.1175254	0.1456640	0.1721380
0.09	0.7	0.0116012	0.0213357	0.0344979	0.0511272	0.0708750	0.0930306	0.1167163	0.1411067
0.09	0.8	0.0129694	0.0239134	0.0387164	0.0573271	0.0791992	0.1033994	0.1288939	0.1547948
0.09	0.9	0.0144690	0.0267425	0.0433290	0.0640374	0.0880743	0.1142704	0.1414593	0.1673804
0.09	1.0	0.0161062	0.0298328	0.0483388	0.0712361	0.0974390	0.1255459	0.1542928	0.1770559
0.10	0.7	0.0128441	0.0235318	0.0378353	0.0556681	0.0765402	0.0996422	0.1240674	0.1490114
0.10	0.8	0.0143521	0.0263477	0.0423865	0.0622592	0.0852623	0.1103744	0.1365551	0.1620906
0.10	0.9	0.0160028	0.0294305	0.0473426	0.0693565	0.0945123	0.1215725	0.1493898	0.1720062
0.10	1.0	0.0178024	0.0327884	0.0527025	0.0769310	0.1042235	0.1331377	0.1618164	0.1819205
0.11	0.7	0.0140786	0.0256983	0.0410990	0.0600673	0.0819836	0.1059586	0.1310677	0.1563072
0.11	0.8	0.0157240	0.0287452	0.0459665	0.0670238	0.0910737	0.1170267	0.1438451	0.1664312
0.11	0.9	0.0175231	0.0320729	0.0512477	0.0744808	0.1006696	0.1285281	0.1567588	0.1765861
0.11	1.0	0.0194820	0.0356879	0.0569366	0.0824032	0.1107004	0.1403631	0.1661510	0.1867368
0.12	0.7	0.0153047	0.0278364	0.0442938	0.0643372	0.0872295	0.1120159	0.1377632	0.1603691
0.12	0.8	0.0170856	0.0311074	0.0494630	0.0716370	0.0966623	0.1233973	0.1508131	0.1707322
0.12	0.9	0.0190305	0.0346718	0.0550524	0.0794302	0.1065800	0.1351822	0.1608327	0.1811242
0.12	1.0	0.0211454	0.0385343	0.0610521	0.0876768	0.1169079	0.1472704	0.1704461	0.1915093

affine transformation $\mu_e = \dfrac{3 - 2\alpha^2}{\alpha^2}$. By this substitution, the following are obtained.

$$V = 6\, q\, L_S \left[\frac{\sqrt{3 - 2\alpha^2}}{\sqrt{3 - 2\alpha^2} + \alpha^2} \right]$$

$$\begin{aligned} M_S &= 3\, q\, L_S^2 \,(3 - 2\alpha^2) && \text{Short direction} \\ M_L &= 3\, q\, L_S^2 \alpha^2 && \text{Long direction} \end{aligned} \qquad (3.35.25)$$

The ultimate axial load capacity of the bearing wall, assuming an equivalent static load is applied to the orthotropic slab, is

L_S loading
$$\frac{q}{f'_{dc}} = \left[\frac{0.1575 + 0.333\, q_d}{\dfrac{L_S}{d_{wall}} \left(\dfrac{\sqrt{3 - 2\alpha^2}}{\sqrt{3 - 2\alpha^2} + \alpha^2} \right) + 0.185} \right] \qquad (3.35.26a)$$

L_L loading
$$\frac{q}{f'_{dc}} = \left[\frac{0.1575 + 0.333\, q_d}{\dfrac{L_S}{d_{wall}} \left(\dfrac{\alpha}{\sqrt{3 - 2\alpha^2} + \alpha^2} \right) + 0.185} \right] \qquad (3.35.26b)$$

Assuming a balanced equivalent loading ($e_d = e_{db}$, $P_{du} = P_{db}$), the equations for ultimate capacity of the bearing wall are

L_S loading
$$\frac{q}{f'_{dc}} = \frac{10{,}830}{\left[\dfrac{L_S}{d_{wall}} \left(\dfrac{\sqrt{3 - 2\alpha^2}}{\sqrt{3 - 2\alpha^2} + \alpha^2} \right) + 0.185 \right] \left[90{,}000 + f_{dy} \right]} \qquad (3.35.27a)$$

L_L loading
$$\frac{q}{f'_{dc}} = \frac{10{,}830}{\left[\dfrac{L_S}{d_{wall}} \left(\dfrac{\alpha}{\sqrt{3 - 2\alpha^2} + \alpha^2} \right) + 0.185 \right] \left[90{,}000 + f_{dy} \right]} \qquad (3.35.27b)$$

Assuming a tensile failure due to flexural loading ($e_d \geq e_{db}$, $P_{du} \leq P_{db}$), the ultimate equivalent load capacity of the bearing wall is

L_S loading

$$\frac{q}{f'_{dc}} = \left[\frac{0.1417}{\frac{L_S}{d}\left(\frac{\sqrt{3-2\alpha^2}}{\sqrt{3-2\alpha^2}+\alpha^2}\right)+0.185}\right] \times \left[\left(0.555-\frac{M_{du}}{d\,P_{du}}\right)+\sqrt{\left(0.555-\frac{M_{du}}{d\,P_{du}}\right)^2+2.09\,q_d}\;\right] \tag{3.35.28a}$$

where

$$\frac{M_{du}}{P_{du}\,d_{wall}} = \left(\frac{1.5\,L_S}{d_{wall}}\right)\frac{\left[\left(\frac{3-2\alpha^2}{3}\right)+0.370\left(\frac{\sqrt{3-2\alpha^2}}{\sqrt{3-2\alpha^2}+\alpha^2}\right)(d/L_S)\right]}{\left[\left(\frac{\sqrt{3-2\alpha^2}}{\sqrt{3-2\alpha^2}+\alpha^2}\right)+0.185\,d/L_S\right]} \tag{3.35.28b}$$

L_L loading

$$\frac{q}{f'_{dc}} = \left[\frac{0.1417}{\frac{L_S}{d}\left(\frac{\alpha}{\sqrt{3-2\alpha^2}+\alpha^2}\right)+0.185}\right] \times \left[\left(0.555-\frac{M_{du}}{d\,P_{du}}\right)+\sqrt{\left(0.555-\frac{M_{du}}{d\,P_{du}}\right)^2+2.09\,q_d}\;\right] \tag{3.35.29a}$$

where

$$\frac{M_{du}}{P_{du}\,d_{wall}} = \left(\frac{1.5\,L_S}{d_{wall}}\right)\left[\frac{\frac{\alpha^2}{3}+\left(\frac{0.370\alpha}{\sqrt{3-2\alpha^2}+\alpha^2}\right)d/L_S}{\left(\frac{\alpha}{\sqrt{3-2\alpha^2}+\alpha^2}\right)+0.185\,d/L_S}\right] \tag{3.35.29b}$$

The ultimate equivalent load capacity of the bearing wall, assuming a compressive failure in flexure ($e_d \leq e_{db}$, $P_{du} \geq P_{db}$) is

$$L_S \text{ direction} \quad \frac{q}{f'_{dc}} = \left[\frac{0.1575 + 0.333\, q_d}{\frac{L_S}{d}\left(\frac{\sqrt{3 - 2\alpha^2}}{\sqrt{3 - 2\alpha^2} + \alpha^2} \right) + 0.185} \right]$$

$$\times \left[\frac{1}{1 + \left\{ \frac{(0.945 + 2\, q_d)\,(90{,}000 + f_{dy})}{65{,}000} - 1 \right\} \frac{e_d}{e_{db}}} \right]$$

(3.35.30)

$$L_L \text{ direction} \quad \frac{q}{f'_{dc}} = \left[\frac{0.1575 + 0.333\, q_d}{\frac{L_S}{d}\left(\frac{\alpha}{\sqrt{3 - 2\alpha^2} + \alpha^2} \right) + 0.185} \right]$$

$$\times \left[\frac{1}{1 + \left\{ \frac{(0.945 + 2\, q_d)\,(90{,}000 + f_{dy})}{65{,}000} - 1 \right\} \frac{e_d}{e_{db}}} \right]$$

(3.35.31)

Tables 3.49 and 3.56 supply computed resistance functions as obtained for eccentrically-loaded bearing walls, with side and end walls separately considered, and equivalent static loading supplied by two-way orthotropic reinforced concrete slabs.

(4) Cost Studies

Giving consideration to the most economical use of materials, it becomes apparent that only tension-type wall failures are of importance for the spans and equivalent loadings considered in this study. This type of failure is expressed, in its most general form, by Equation 3.35.6. For walls or columns supporting one-way reinforced slabs (Tables 3-41 to 3-44), tension governs for $\frac{q}{f'_{dc}} < 0.09482$.

This value corresponds to the following pressures and concrete ratios.

Table 3-49

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS, SHORT-SPAN DIRECTION (f_{dy} = 44,000 psi)

q_d		Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft							
Wall	α	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
0.03	0.7	0.0026045	0.0048182	0.0079224	0.0121355	0.0177402	0.0250631	0.0344104	0.0459543
0.03	0.8	0.0030872	0.0057525	0.0095468	0.0147853	0.0218695	0.0312309	0.0431897	0.0577646
0.03	0.9	0.0039068	0.0073668	0.0124115	0.0195549	0.0294111	0.0425036	0.0589380	0.0781996
0.03	1.0	0.0055500	0.0107031	0.0185324	0.0300151	0.0460059	0.0665447	0.0905356	0.1163519
0.04	0.7	0.0034642	0.0063911	0.0104615	0.0159122	0.0230169	0.0320382	0.0431438	0.0563152
0.04	0.8	0.0041043	0.0076222	0.0125781	0.0193033	0.0281625	0.0394557	0.0532742	0.0693937
0.04	0.9	0.0051894	0.0097408	0.0162795	0.0253127	0.0373255	0.0525492	0.0707385	0.0911843
0.04	1.0	0.0073574	0.0140811	0.0240425	0.0380591	0.0565262	0.0789397	0.1040062	0.1302794
0.05	0.7	0.0043197	0.0079482	0.0129544	0.0195752	0.0280509	0.0385597	0.0511376	0.0656238
0.05	0.8	0.0051156	0.0096694	0.0155424	0.0236548	0.0341026	0.0470424	0.0623769	0.0797275
0.05	0.9	0.0064626	0.0120771	0.0200332	0.0307871	0.0446622	0.0616326	0.0812205	0.1026301
0.05	1.0	0.0091448	0.0173760	0.0292983	0.0455087	0.0659951	0.0899034	0.1158758	0.1426135
0.06	0.7	0.0051711	0.0094900	0.0154036	0.0231343	0.0328727	0.0447060	0.0585531	0.0741476
0.06	0.8	0.0061212	0.0112949	0.0184438	0.0278569	0.0397434	0.0541197	0.0707385	0.0891214
0.06	0.9	0.0077266	0.0143776	0.0236821	0.0360162	0.0515318	0.0699867	0.0907467	0.1129812
0.06	1.0	0.0109128	0.0205937	0.0343321	0.0524789	0.0746762	0.0998403	0.1266076	0.1538012
0.07	0.7	0.0060184	0.0110170	0.0178113	0.02659[illegible]8	0.0375073	0.0505354	0.0655000	0.0820570
0.07	0.8	0.0071211	0.0130995	0.0212864	0.0319241	0.0451259	0.0607783	0.0785151	0.0977931
0.07	0.9	0.0089816	0.0[illegible]66439	0.0272345	0.0410304	0.0580136	0.0777632	0.0995389	0.1225022
0.07	1.0	0.0126620	0.0237394	0.0391699	0.0590519	0.0827387	0.1089963	0.1364772	0.1641128
0.08	0.7	0.0068618	0.0125296	0.0201796	0.0299730	0.0419747	0.0560922	0.0720574	0.0894682
0.08	0.8	0.0081155	0.0148839	0.0240734	0.0358685	0.0502827	0.0670847	0.0858145	0.1058873
0.08	0.9	0.0102277	0.0188774	0.0306977	0.0458540	0.0641667	0.0850675	0.1077445	0.1313655
0.08	1.0	0.0143929	0.0268177	0.0438329	0.0652886	0.0902983	0.1175254	0.1456640	0.1737267
0.09	0.7	0.0077012	0.0140281	0.0225102	0.0332663	0.0462920	0.0614115	0.0782842	0.0964651
0.09	0.8	0.0091046	0.0166487	0.0268080	0.0397007	0.0552400	0.0730896	0.0927152	0.1135059
0.09	0.9	0.0114651	0.0210796	0.0340782	0.0505074	0.0700364	0.0919765	0.1154672	0.1396913
0.09	1.0	0.0161062	0.0298328	0.0483388	0.0712351	0.0974390	0.1255459	0.1542928	0.1827679
0.10	0.7	0.0085368	0.0155130	0.0248050	0.0364835	0.0504733	0.0665214	0.0842259	0.1031101
0.10	0.8	0.0100882	0.0183946	0.0794932	0.0434297	0.0600193	0.0788323	0.0992762	0.1207241
0.10	0.9	0.0126941	0.0232516	0.0373815	0.0550074	0.0756588	0.0985481	0.1227834	0.1475667
0.10	1.0	0.0178024	0.0327884	0.0527025	0.0769310	0.1042235	0.1331377	0.1624543	0.1913279
0.11	0.7	0.0093685	0.0169847	0.0270655	0.0396295	0.0545306	0.0714448	0.0899184	0.1094517
0.11	0.8	0.0110666	0.0201221	0.0321314	0.0470634	0.0646386	0.0843446	0.1055430	0.1275991
0.11	0.9	0.0139147	0.0253948	0.0406127	0.0593681	0.0810626	0.1048274	0.1297512	0.1550579
0.11	1.0	0.0194820	0.0356679	0.0569366	0.0824032	0.1107004	0.1403631	0.1702171	0.1994763
0.12	0.7	0.0101965	0.0184434	0.0292933	0.0427090	0.0584745	0.0762008	0.0953907	0.1155278
0.12	0.8	0.0120398	0.0218319	0.0347251	0.0506089	0.0691120	0.0896521	0.1115522	0.1341756
0.12	0.9	0.0151272	0.0275102	0.0437764	0.0636019	0.0862716	0.1108503	0.1364162	0.1622161
0.12	1.0	0.0211454	0.0385343	0.0610521	0.0876768	0.1169079	0.1472704	0.1776345	0.2072673

Table 3-50

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS, SHORT-SPAN DIRECTION (f_{dy} = 52, 000 psi)

q_d		Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft							
Wall	α	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
0.03	0.7	0.0026045	0.0048182	0.0079224	0.0121355	0.0177402	0.0250631	0.0344104	0.0459543
0.03	0.8	0.0030872	0.0057525	0.0095468	0.0147853	0.0218695	0.0312309	0.0431897	0.0577646
0.03	0.9	0.0039068	0.0073668	0.0124115	0.0195549	0.0294111	0.0425036	0.0589380	0.0781996
0.03	1.0	0.0055500	0.0107031	0.0185324	0.0300151	0.0460059	0.0665447	0.0905356	0.1163519
0.04	0.7	0.0034642	0.0063911	0.0104615	0.0159122	0.0230169	0.0320382	0.0431438	0.0563152
0.04	0.8	0.0041043	0.0076222	0.0125781	0.0193033	0.0281625	0.0394557	0.0532742	0.0693937
0.04	0.9	0.0051894	0.0097408	0.0162795	0.0253127	0.0373255	0.0525492	0.0707385	0.0911843
0.04	1.0	0.0073574	0.0140811	0.0240425	0.0380591	0.0565262	0.0789397	0.1040062	0.1302794
0.05	0.7	0.0043197	0.0079482	0.0129544	0.0195752	0.0280509	0.0385597	0.0511376	0.0656238
0.05	0.8	0.0051156	0.0094696	0.0155426	0.0236548	0.0341026	0.0470424	0.0623769	0.0797275
0.05	0.9	0.0064626	0.0120771	0.0200332	0.0307871	0.0446622	0.0616326	0.0812205	0.1026301
0.05	1.0	0.0091448	0.0173760	0.0292983	0.0455087	0.0659951	0.0899034	0.1158758	0.1426135
0.06	0.7	0.0051711	0.0094900	0.0154036	0.0231343	0.0328727	0.0447060	0.0585531	0.0741476
0.06	0.8	0.0061212	0.0112949	0.0184438	0.0278569	0.0397434	0.0541197	0.0707385	0.0891214
0.06	0.9	0.0077266	0.0143776	0.0236821	0.0360162	0.0515318	0.0699867	0.0907467	0.1129812
0.06	1.0	0.0109128	0.0205937	0.0343321	0.0524789	0.0746762	0.0998403	0.1266076	0.1538012
0.07	0.7	0.0060184	0.0110170	0.0178113	0.0265978	0.0375073	0.0505354	0.0655000	0.0820570
0.07	0.8	0.0071211	0.0130995	0.0212864	0.0319241	0.0451259	0.0607783	0.0785151	0.0977031
0.07	0.9	0.0089816	0.0166439	0.0272345	0.0410304	0.0580136	0.0777632	0.0995389	0.1225022
0.07	1.0	0.0126620	0.0237394	0.0391699	0.0590519	0.0827387	0.1089963	0.1364772	0.1641128
0.08	0.7	0.0068618	0.0125296	0.0201796	0.0299730	0.0419747	0.0560922	0.0720574	0.0894682
0.08	0.8	0.0081155	0.0148839	0.0240734	0.0358685	0.0502827	0.0670847	0.0858145	0.1058873
0.08	0.9	0.0102277	0.0188774	0.0306977	0.0458540	0.0641667	0.0850675	0.1077445	0.1313655
0.08	1.0	0.0143929	0.0268177	0.0438329	0.0652886	0.0902983	0.1175254	0.1456640	0.1737267
0.09	0.7	0.0077012	0.0140281	0.0225102	0.0332663	0.0462920	0.0614115	0.0782842	0.0964651
0.09	0.8	0.0091046	0.0166487	0.0268080	0.0397007	0.0552400	0.0730896	0.0927152	0.1135059
0.09	0.9	0.0114651	0.0210796	0.0340782	0.0505074	0.0700364	0.0919765	0.1154672	0.1396913
0.09	1.0	0.0161062	0.0298328	0.0483388	0.0712361	0.0974390	0.1255459	0.1542928	0.1827679
0.10	0.7	0.0085368	0.0155130	0.0248050	0.0364835	0.0504733	0.0665214	0.0842259	0.1031101
0.10	0.8	0.0100882	0.0183946	0.0294932	0.0434297	0.0600193	0.0788323	0.0992762	0.1207241
0.10	0.9	0.0126941	0.0232516	0.0373815	0.0550074	0.0756588	0.0985481	0.1227834	0.1475667
0.10	1.0	0.0178024	0.0327884	0.0527025	0.0769310	0.1042235	0.1331377	0.1624543	0.1913279
0.11	0.7	0.0093685	0.0169847	0.0270655	0.0396295	0.0545306	0.0714448	0.0899184	0.1094517
0.11	0.8	0.0110666	0.0201221	0.0321314	0.0470634	0.0646386	0.0843446	0.1055430	0.1275991
0.11	0.9	0.0139147	0.0253948	0.0406127	0.0593681	0.0810626	0.1048274	0.1297512	0.1550579
0.11	1.0	0.0194820	0.0356879	0.0569366	0.0824032	0.1107004	0.1403631	0.1702171	0.1988230
0.12	0.7	0.0101965	0.0184434	0.0292933	0.0427090	0.0584745	0.0762008	0.0953907	0.1155278
0.12	0.8	0.0120398	0.0218319	0.0347251	0.0506089	0.0691128	0.0896521	0.1115522	0.1341756
0.12	0.9	0.0151272	0.0275102	0.0437764	0.0636019	0.0862716	0.1108503	0.1364162	0.1622161
0.12	1.0	0.0211454	0.0385343	0.0610521	0.0876768	0.1169079	0.1472704	0.1776345	0.2034903

Table 3-51

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS, SHORT-SPAN DIRECTION (f_{dy} = 60,000 psi)

q_d		Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft							
Wall	α	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
0.03	0.7	0.0026045	0.0048182	0.0079224	0.0121355	0.0177402	0.0250631	0.0344104	0.0459563
0.03	0.8	0.0030872	0.0057525	0.0095468	0.0147853	0.0218695	0.0312309	0.0431897	0.0577646
0.03	0.9	0.0039068	0.0073668	0.0124115	0.0195549	0.0294111	0.0425036	0.0589380	0.0781996
0.03	1.0	0.0055500	0.0107031	0.0185324	0.0300151	0.0460059	0.0665447	0.0905356	0.1163519
0.04	0.7	0.0034642	0.0063911	0.0104615	0.0159122	0.0230169	0.0320382	0.0431438	0.0563152
0.04	0.8	0.0041043	0.0076222	0.0125781	0.0193033	0.0281625	0.0394557	0.0532742	0.0693937
0.04	0.9	0.0051894	0.0097408	0.0162795	0.0253127	0.0373255	0.0525492	0.0707385	0.0911843
0.04	1.0	0.0073574	0.0140811	0.0240425	0.0380591	0.0565262	0.0789397	0.1040062	0.1302794
0.05	0.7	0.0043197	0.0079482	0.0129544	0.0195752	0.0280509	0.0385597	0.0511376	0.0656238
0.05	0.8	0.0051156	0.0094694	0.0155426	0.0236548	0.0341026	0.0470624	0.0623769	0.0797275
0.05	0.9	0.0064626	0.0120771	0.0200332	0.0307871	0.0446622	0.0616326	0.0812205	0.1026301
0.05	1.0	0.0091448	0.0173760	0.0292983	0.0455087	0.0659951	0.0899034	0.1158758	0.1426135
0.06	0.7	0.0051711	0.0094900	0.0154036	0.0231343	0.0328727	0.0447060	0.0585531	0.0741476
0.06	0.8	0.0061212	0.0112949	0.0184438	0.0278569	0.0397434	0.0541197	0.0707385	0.0891214
0.06	0.9	0.0077266	0.0143776	0.0236821	0.0360162	0.0515318	0.0699867	0.0907467	0.1129812
0.06	1.0	0.0109128	0.0205937	0.0343321	0.0524789	0.0746762	0.0998403	0.1266076	0.1538012
0.07	0.7	0.0060184	0.0110170	0.0178113	0.0265978	0.0375073	0.0505354	0.0655000	0.0820570
0.07	0.8	0.0071211	0.0130995	0.0212864	0.0319241	0.0451259	0.0607783	0.0785151	0.0977931
0.07	0.9	0.0089816	0.0166439	0.0272345	0.0410304	0.0580136	0.0777632	0.0995389	0.1225022
0.07	1.0	0.0126620	0.0237394	0.0391699	0.0590519	0.0827387	0.1089943	0.1364772	0.1641128
0.08	0.7	0.0069618	0.0125296	0.0201796	0.0299730	0.0419747	0.0560922	0.0720574	0.0894682
0.08	0.8	0.0081155	0.0148839	0.0240734	0.0358685	0.0502827	0.0670847	0.0858145	0.1058873
0.08	0.9	0.0102277	0.0188774	0.0306977	0.0458540	0.0641667	0.0850675	0.1077445	0.1313655
0.08	1.0	0.0143929	0.0268177	0.0438329	0.0652886	0.0902983	0.1175254	0.1455640	0.1737267
0.09	0.7	0.0077012	0.0140281	0.0225102	0.0332663	0.0462920	0.0614115	0.0782842	0.0964651
0.09	0.8	0.0091046	0.0166487	0.0268080	0.0397007	0.0552400	0.0730896	0.0927152	0.1135059
0.09	0.9	0.0114651	0.0210796	0.0340782	0.0505074	0.0700364	0.0919765	0.1154672	0.1396913
0.09	1.0	0.0161062	0.0298328	0.0483388	0.0712361	0.0974390	0.1255459	0.1542928	0.1827679
0.10	0.7	0.0085368	0.0155130	0.0248050	0.0364835	0.0504733	0.0665214	0.0842259	0.1031101
0.10	0.8	0.0100882	0.0183946	0.0294932	0.0434297	0.0600193	0.0788323	0.0992762	0.1207241
0.10	0.9	0.0126941	0.0232516	0.0373815	0.0550074	0.0756588	0.0985481	0.1227834	0.1475667
0.10	1.0	0.0178024	0.0327884	0.0527025	0.0769310	0.1042235	0.1331377	0.1624543	0.1896110
0.11	0.7	0.0093685	0.0169847	0.0270655	0.0396295	0.0545306	0.0714448	0.0899184	0.1094517
0.11	0.8	0.0110666	0.0201221	0.0321314	0.0470634	0.0646386	0.0843446	0.1055430	0.1275991
0.11	0.9	0.0139147	0.0253948	0.0406127	0.0593681	0.0810626	0.1048274	0.1297512	0.1550579
0.11	1.0	0.0194820	0.0356879	0.0569366	0.0824032	0.1107004	0.1403631	0.1702171	0.1943594
0.12	0.7	0.0101965	0.0184434	0.0292933	0.0427090	0.0584745	0.0762008	0.0953907	0.1155278
0.12	0.8	0.0120398	0.0218319	0.0347251	0.0506089	0.0691128	0.0896521	0.1115522	0.1341756
0.12	0.9	0.0151272	0.0275102	0.0437764	0.0636019	0.0862716	0.1108503	0.1364162	0.1622161
0.12	1.0	0.0211454	0.0385343	0.0610521	0.0876768	0.1169079	0.1472704	0.1774170	0.1990668

Table 3-52

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS, SHORT-SPAN DIRECTION (f_{dy} = 75, 000 psi)

q_d		Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft							
Wall	α	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
0.03	0.7	0.0026045	0.0048182	0.0079224	0.0121355	0.0177402	0.0250631	0.0344104	0.0459543
0.03	0.8	0.0030872	0.0057525	0.0095468	0.0147853	0.0218695	0.0312309	0.0431897	0.0577646
0.03	0.9	0.0039068	0.0073668	0.0124115	0.0195549	0.0294111	0.0425036	0.0589380	0.0781996
0.03	1.0	0.0055500	0.0107031	0.0185324	0.0300151	0.0460059	0.0665447	0.0905356	0.1163519
0.04	0.7	0.0034642	0.0063911	0.0104615	0.0159122	0.0230169	0.0320382	0.0431438	0.0563152
0.04	0.8	0.0041043	0.0076222	0.0125781	0.0193033	0.0281625	0.0394557	0.0532742	0.0693937
0.04	0.9	0.0051894	0.0097408	0.0162795	0.0253127	0.0373255	0.0525492	0.0707385	0.0911843
0.04	1.0	0.0073574	0.0140811	0.0240425	0.0380591	0.0565262	0.0789397	0.1040062	0.1302794
0.05	0.7	0.0043197	0.0079482	0.0129544	0.0195752	0.0280509	0.0385597	0.0511376	0.0656238
0.05	0.8	0.0051156	0.0094694	0.0155424	0.0236548	0.0341026	0.0470424	0.0623769	0.0797275
0.05	0.9	0.0064626	0.0120771	0.0200332	0.0307871	0.0446622	0.0616326	0.0812205	0.1026301
0.05	1.0	0.0091448	0.0173760	0.0292983	0.0455087	0.0659951	0.0899034	0.1158758	0.1426135
0.06	0.7	0.0051711	0.0094900	0.0154036	0.0231343	0.0328727	0.0447060	0.0585531	0.0741476
0.06	0.8	0.0061212	0.0112949	0.0184438	0.0278569	0.0397434	0.0541197	0.0707385	0.0891214
0.06	0.9	0.0077256	0.0143776	0.0236821	0.0360162	0.0515318	0.0699867	0.0907467	0.1129812
0.06	1.0	0.0109128	0.0205937	0.0343321	0.0524789	0.0746762	0.0998403	0.1266076	0.1538012
0.07	0.7	0.0060184	0.0110170	0.0178113	0.0265978	0.0375073	0.0505354	0.0655000	0.0820570
0.07	0.8	0.0071211	0.0130995	0.0212864	0.0319241	0.0451259	0.0607783	0.0785151	0.0977931
0.07	0.9	0.0089816	0.0166439	0.0272345	0.0410304	0.0580136	0.0777632	0.0995389	0.1225022
0.07	1.0	0.0126620	0.0237394	0.0391699	0.0590519	0.0827337	0.1089943	0.1364772	0.1641128
0.08	0.7	0.0068618	0.0125296	0.0201796	0.0299730	0.0419747	0.0560922	0.0720574	0.0894682
0.08	0.8	0.0081155	0.0148839	0.0240734	0.0358685	0.0502827	0.0670847	0.0858145	0.1058873
0.08	0.9	0.0102277	0.0188774	0.0306977	0.0458540	0.0641667	0.0850675	0.1077445	0.1313655
0.08	1.0	0.0143929	0.0268177	0.0438329	0.0652886	0.0902983	0.1175254	0.1456640	0.1721380
0.09	0.7	0.0077012	0.0140281	0.0225102	0.0332663	0.0462920	0.0614115	0.0782842	0.0954651
0.09	0.8	0.0091046	0.0166487	0.0268080	0.0397007	0.0552400	0.0730896	0.0927152	0.1135059
0.09	0.9	0.0114651	0.0210796	0.0340782	0.0505074	0.0700364	0.0919765	0.1154672	0.1396913
0.09	1.0	0.0161062	0.0298328	0.0483388	0.0712361	0.0974390	0.1255459	0.1542928	0.1770559
0.10	0.7	0.0085368	0.0155130	0.0248050	0.0364835	0.0504733	0.0665214	0.0842259	0.1031101
0.10	0.8	0.0100882	0.0183946	0.0294932	0.0434297	0.0600193	0.0788323	0.0992762	0.1207241
0.10	0.9	0.0126941	0.0232516	0.0373815	0.0550074	0.0756588	0.0985481	0.1227834	0.1475667
0.10	1.0	0.0178024	0.0327884	0.0527025	0.0769310	0.1042235	0.1331377	0.1618164	0.1819205
0.11	0.7	0.0093685	0.0169847	0.0270655	0.0396295	0.0545306	0.0714448	0.0899184	0.1094517
0.11	0.8	0.0110666	0.0201121	0.0321314	0.0470634	0.0646386	0.0843446	0.1055430	0.1275991
0.11	0.9	0.0139147	0.0253948	0.0406127	0.0593681	0.0810626	0.1048274	0.1297512	0.1550579
0.11	1.0	0.0194820	0.0356879	0.0569366	0.0824032	0.1107004	0.1403631	0.1661510	0.1867368
0.12	0.7	0.0101965	0.0184434	0.0292933	0.0427090	0.0584745	0.0762008	0.0953907	0.1155278
0.12	0.8	0.0120398	0.0218319	0.0347251	0.0506089	0.0691128	0.0896521	0.1115522	0.1341756
0.12	0.9	0.0151272	0.0275102	0.0437764	0.0636019	0.0862716	0.1108503	0.1364162	0.1592861
0.12	1.0	0.0211454	0.0385343	0.0610521	0.0876768	0.1169079	0.1472704	0.1704461	0.1915093

Table 3-53

RESISTANCE FUNCTIONS d/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS, LONG-SPAN DIRECTION (f_{dy} = 44,000 psi)

q_d		Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft							
Wall	α	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
0.03	0.7	0.0125840	0.0260858	0.0478993	0.0780333	0.1130005	0.1489860	0.1838128	0.2165891
0.03	0.8	0.0091866	0.0184785	0.0333731	0.0552086	0.0831801	0.1145336	0.1466280	0.1778946
0.03	0.9	0.0070204	0.0137891	0.0243651	0.0400596	0.0614248	0.0873762	0.1157833	0.1447514
0.03	1.0	0.0055500	0.0107031	0.0185324	0.0300151	0.0460059	0.0665447	0.0905356	0.1163519
0.04	0.7	0.0164870	0.0332749	0.0586862	0.0908048	0.1266146	0.1627430	0.1974677	0.2300839
0.04	0.8	0.0121105	0.0239562	0.0419779	0.0666138	0.0962708	0.1282915	0.1605107	0.1916889
0.04	0.9	0.0092873	0.0180429	0.0312397	0.0497638	0.0734224	0.1007067	0.1296579	0.1587370
0.04	1.0	0.0073574	0.0140811	0.0240425	0.0380591	0.0565262	0.0789397	0.1040062	0.1302794
0.05	0.7	0.0202676	0.0399791	0.0679716	0.1020604	0.1386683	0.1750714	0.2098532	0.2424488
0.05	0.8	0.0149726	0.0291759	0.0498791	0.0767691	0.1078063	0.1404739	0.1729354	0.2041706
0.05	0.9	0.0115204	0.0221545	0.0376917	0.0585725	0.0840618	0.1124513	0.1419447	0.1712408
0.05	1.0	0.0091448	0.0173760	0.0292983	0.0455087	0.0659951	0.0899034	0.1158758	0.1426135
0.06	0.7	0.0239365	0.0462850	0.0766454	0.1122383	0.1495999	0.1863410	0.2212688	0.2495311
0.06	0.8	0.0177768	0.0341710	0.0572254	0.0860130	0.1162315	0.1515230	0.1842831	0.2156548
0.06	0.9	0.0137211	0.0261373	0.0437905	0.0666956	0.0937208	0.1230694	0.1530891	0.1826530
0.06	1.0	0.0109128	0.0205937	0.0343321	0.0524789	0.0746762	0.0998403	0.1266076	0.1538012
0.07	0.7	0.0275031	0.0522561	0.0846846	0.1215992	0.1596746	0.1967853	0.2319120	0.2560074
0.07	0.8	0.0205263	0.0389682	0.0641195	0.0945540	0.1278205	0.1617064	0.1947933	0.2263488
0.07	0.9	0.0158908	0.0300028	0.0495886	0.0742719	0.1026289	0.1328338	0.1633607	0.1932175
0.07	1.0	0.0126520	0.0237394	0.0391699	0.0590519	0.0827387	0.1089963	0.1364772	0.1641128
0.08	0.7	0.0309756	0.0579408	0.0922110	0.1303130	0.1690668	0.2065627	0.2379059	0.2624116
0.08	0.8	0.0232243	0.0435893	0.0706359	0.1025317	0.1367458	0.1712003	0.2046278	0.2343145
0.08	0.9	0.0180308	0.0337607	0.0551264	0.0813990	0.1109380	0.1419223	0.1729372	0.2030993
0.08	1.0	0.0143929	0.0268177	0.0438329	0.0652886	0.0902983	0.1175254	0.1456640	0.1737267
0.09	0.7	0.0343609	0.0633767	0.0993118	0.1384978	0.1778990	0.2157867	0.2437175	0.2687516
0.09	0.8	0.0258735	0.0480523	0.0768305	0.1100448	0.1451286	0.1801284	0.2139023	0.2400526
0.09	0.9	0.0201422	0.0374195	0.0604361	0.0881484	0.1187548	0.1504585	0.1819431	0.2124158
0.09	1.0	0.0161062	0.0298328	0.0483388	0.0712361	0.0974390	0.1255459	0.1542928	0.1827679
0.10	0.7	0.0376654	0.0685937	0.1060519	0.1462397	0.1862608	0.2224458	0.2494755	0.2750352
0.10	0.8	0.0284766	0.0523726	0.0827468	0.1171659	0.1530574	0.1885812	0.2215632	0.2457376
0.10	0.9	0.0222262	0.0409867	0.0655438	0.0945744	0.1261576	0.1585323	0.1904696	0.2208934
0.10	1.0	0.0178024	0.0327884	0.0527025	0.0769310	0.1042235	0.1331377	0.1624543	0.1913279
0.11	0.7	0.0408946	0.0736165	0.1124813	0.1536036	0.1942202	0.2275938	0.2551852	0.2812652
0.11	0.8	0.0310357	0.0565628	0.0884193	0.1239508	0.1605987	0.1966272	0.2266971	0.2513747
0.11	0.9	0.0242839	0.0444689	0.0704710	0.1007195	0.1332061	0.1662115	0.1985860	0.2260166
0.11	1.0	0.0194820	0.0356879	0.0569366	0.0824032	0.1107004	0.1403631	0.1702171	0.1994763
0.12	0.7	0.0440533	0.0784652	0.1186394	0.1606400	0.2018302	0.2327019	0.2608514	0.2874497
0.12	0.8	0.0335532	0.0606342	0.0938758	0.1304430	0.1678043	0.2043203	0.2317908	0.2569687
0.12	0.9	0.0263161	0.0478719	0.0752355	0.1066176	0.1399466	0.1735489	0.2063465	0.2310999
0.12	1.0	0.0211454	0.0385343	0.0610521	0.0876768	0.1169079	0.1472704	0.1776345	0.2072673

Table 3-54

RESISTANCE FUNCTIONS d/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS, LONG-SPAN DIRECTION (f_{dy} = 52,000 psi)

q_d		Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft							
Wall	α	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
0.03	0.7	0.0125840	0.0260858	0.0478993	0.0780333	0.1130005	0.1489860	0.1838128	0.2165891
0.03	0.8	0.0091866	0.0184785	0.0333731	0.0552086	0.0831801	0.1145336	0.1466280	0.1778946
0.03	0.9	0.0070204	0.0137891	0.0243651	0.0400596	0.0614248	0.0873762	0.1157833	0.1447514
0.03	1.0	0.0055500	0.0107031	0.0185324	0.0300151	0.0460059	0.0665447	0.0905356	0.1163519
0.04	0.7	0.0164870	0.0332749	0.0584842	0.0908048	0.1266146	0.1627430	0.1974677	0.2300839
0.04	0.8	0.0121105	0.0239562	0.0419779	0.0666138	0.0962708	0.1282915	0.1605197	0.1916889
0.04	0.9	0.0092873	0.0180429	0.0312397	0.0497638	0.0734224	0.1007067	0.1296579	0.1587370
0.04	1.0	0.0073574	0.0140811	0.0240425	0.0380591	0.0565262	0.0789397	0.1040062	0.1302794
0.05	0.7	0.0202676	0.0399791	0.0679716	0.1020604	0.1386683	0.1750714	0.2098532	0.2367586
0.05	0.8	0.0149726	0.0291759	0.0498791	0.0767691	0.1078043	0.1404739	0.1729354	0.2041704
0.05	0.9	0.0115204	0.0221545	0.0376917	0.0585725	0.0840618	0.1124513	0.1419447	0.1712408
0.05	1.0	0.0091448	0.0173760	0.0292983	0.0455087	0.0659951	0.0899034	0.1158758	0.1426135
0.06	0.7	0.0239365	0.0462850	0.0766454	0.1122383	0.1495999	0.1863410	0.2203099	0.2434032
0.06	0.8	0.0177768	0.0341710	0.0572254	0.0860130	0.1182315	0.1515230	0.1842831	0.2156548
0.06	0.9	0.0137211	0.0261373	0.0437905	0.0666956	0.0937208	0.1230694	0.1530891	0.1826530
0.06	1.0	0.0109128	0.0205937	0.0343321	0.0524789	0.0746762	0.0998403	0.1266076	0.1538012
0.07	0.7	0.0275031	0.0522561	0.0846846	0.1215992	0.1596746	0.1967853	0.2263226	0.2499617
0.07	0.8	0.0205263	0.0389682	0.0641195	0.0945540	0.1278205	0.1617064	0.1947933	0.2228397
0.07	0.9	0.0158908	0.0300028	0.0495986	0.0742719	0.1026289	0.1328338	0.1633607	0.1932175
0.07	1.0	0.0126620	0.0237394	0.0391699	0.0590519	0.0827387	0.1089963	0.1364772	0.1641128
0.08	0.7	0.0309756	0.0579408	0.0922110	0.1303130	0.1690668	0.2065627	0.2322642	0.2564437
0.08	0.8	0.0232243	0.0435893	0.0706359	0.1025317	0.1367458	0.1712003	0.2046278	0.2287052
0.08	0.9	0.0180308	0.0337607	0.0551264	0.0813990	0.1109380	0.1419223	0.1729372	0.2030993
0.08	1.0	0.0143929	0.0268177	0.0438329	0.0652886	0.0902983	0.1175254	0.1456640	0.1737267
0.09	0.7	0.0343609	0.0633767	0.0993118	0.1384978	0.1778990	0.2120577	0.2381424	0.2628574
0.09	0.8	0.0258735	0.0480523	0.0768305	0.1100448	0.1451286	0.1801284	0.2111858	0.2345098
0.09	0.9	0.0201422	0.0374195	0.0604361	0.0881484	0.1187548	0.1504585	0.1819431	0.2105229
0.09	1.0	0.0161062	0.0298328	0.0483388	0.0712361	0.0974390	0.1255459	0.1542928	0.1827579
0.10	0.7	0.0376654	0.0685937	0.1060519	0.1462397	0.1862608	0.2173061	0.2439635	0.2692098
0.10	0.8	0.0284766	0.0523726	0.0827468	0.1171659	0.1530574	0.1885812	0.2164196	0.2402569
0.10	0.9	0.0222262	0.0409867	0.0655438	0.0945744	0.1261576	0.1585323	0.1904696	0.2157458
0.10	1.0	0.0178024	0.0327884	0.0527025	0.0769310	0.1042235	0.1331377	0.1624543	0.1913279
0.11	0.7	0.0408946	0.0736165	0.1124813	0.1536036	0.1938126	0.2225074	0.2497332	0.2755070
0.11	0.8	0.0310357	0.0565628	0.0884193	0.1239508	0.1605987	0.1961084	0.2216064	0.2459532
0.11	0.9	0.0242839	0.0444689	0.0704710	0.1007195	0.1332061	0.1662115	0.1979750	0.2209219
0.11	1.0	0.0194820	0.0356879	0.0569366	0.0824032	0.1107004	0.1403631	0.1702171	0.1988230
0.12	0.7	0.0440533	0.0794652	0.1186394	0.1606400	0.1983637	0.2276662	0.2554566	0.2817543
0.12	0.8	0.0335532	0.0606342	0.0938758	0.1304430	0.1678043	0.2007127	0.2267508	0.2516034
0.12	0.9	0.0263161	0.0478719	0.0752355	0.1066176	0.1399466	0.1735489	0.2026227	0.2260555
0.12	1.0	0.0211454	0.0385343	0.0610521	0.0876768	0.1169079	0.1472704	0.1776345	0.2034903

Table 3-55

RESISTANCE FUNCTIONS d/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS, LONG-SPAN DIRECTION (f_{dy} = 60,000 psi)

q_d Wall	α	Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft: 0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
0.03	0.7	0.0125840	0.0260858	0.0478993	0.0780333	0.1130005	0.1489860	0.1838128	0.2165891
0.03	0.8	0.0091866	0.0184785	0.0333731	0.0552086	0.0831801	0.1145336	0.1466280	0.1778946
0.03	0.9	0.0070204	0.0137891	0.0243651	0.0400596	0.0614248	0.0873762	0.1157833	0.1447514
0.03	1.0	0.0055500	0.0107031	0.0185324	0.0300151	0.0460059	0.0665447	0.0905356	0.1163519
0.04	0.7	0.0164870	0.0332749	0.0584842	0.0908048	0.1266146	0.1627630	0.1974677	0.2240927
0.04	0.8	0.0121105	0.0239562	0.0419779	0.0666138	0.0962708	0.1282915	0.1605107	0.1916889
0.04	0.9	0.0092873	0.0180429	0.0312397	0.0497638	0.0734224	0.1007067	0.1296579	0.1587370
0.04	1.0	0.0073574	0.0140811	0.0240425	0.0380591	0.0565262	0.0789397	0.1040062	0.1302794
0.05	0.7	0.0202676	0.0399791	0.0679716	0.1020604	0.1386683	0.1750714	0.2087189	0.2309187
0.05	0.8	0.0149726	0.0291759	0.0498791	0.0767691	0.1078043	0.1404739	0.1729356	0.2041706
0.05	0.9	0.0115204	0.0221545	0.0376917	0.0585725	0.0840618	0.1124513	0.1419447	0.1712408
0.05	1.0	0.0091448	0.0173760	0.0292983	0.0455087	0.0659951	0.0899034	0.1158758	0.1426135
0.06	0.7	0.0239365	0.0462850	0.0766454	0.1122382	0.1495999	0.1863410	0.2148821	0.2376427
0.06	0.8	0.0177768	0.0341710	0.0572254	0.0860130	0.1182315	0.1515230	0.1842831	0.2115113
0.06	0.9	0.0137211	0.0261373	0.0437905	0.0666956	0.0937208	0.1230694	0.1530891	0.1826530
0.06	1.0	0.0109128	0.0205937	0.0343321	0.0524789	0.0746762	0.0998403	0.1266076	0.1538012
0.07	0.7	0.0275031	0.0522561	0.0846846	0.1215992	0.1596746	0.1964200	0.2209618	0.2442763
0.07	0.8	0.0205263	0.0389682	0.0641195	0.0945540	0.1278205	0.1617064	0.1947933	0.2175133
0.07	0.9	0.0158908	0.0300028	0.0495886	0.0742719	0.1026289	0.1328338	0.1633607	0.1932175
0.07	1.0	0.0126620	0.0237336	0.0391699	0.0590519	0.0827387	0.1089943	0.1364772	0.1641128
0.08	0.7	0.0309756	0.0579408	0.0922110	0.1303130	0.1690668	0.2018360	0.2269670	0.2508294
0.08	0.8	0.0232243	0.0435893	0.0706359	0.1025317	0.1367458	0.1712003	0.2009747	0.2234426
0.08	0.9	0.0180308	0.0337607	0.0551264	0.0813990	0.1109380	0.1419223	0.1729372	0.2003203
0.08	1.0	0.0143929	0.0268177	0.0438329	0.0652886	0.0902983	0.1175254	0.1456640	0.1737267
0.09	0.7	0.0343609	0.0633767	0.0993118	0.1384978	0.1778990	0.2071873	0.2329054	0.2573103
0.09	0.8	0.0258735	0.0480523	0.0768305	0.1100448	0.1451286	0.1801284	0.2053130	0.2293054
0.09	0.9	0.0201422	0.0374195	0.0604361	0.0881484	0.1187548	0.1504585	0.1819431	0.2056474
0.09	1.0	0.0161062	0.0298328	0.0483388	0.0712361	0.0974390	0.1255459	0.1542928	0.1827679
0.10	0.7	0.0376654	0.0685937	0.1060519	0.1462397	0.1848267	0.2126859	0.2387838	0.2637264
0.10	0.8	0.0284766	0.0523726	0.0827468	0.1171659	0.1530574	0.1870189	0.2115967	0.2351087
0.10	0.9	0.0222262	0.0409867	0.0655438	0.0945744	0.1261576	0.1585323	0.1888013	0.2109201
0.10	1.0	0.0178024	0.0327884	0.0527025	0.0769310	0.1042235	0.1331377	0.1624543	0.1896110
0.11	0.7	0.0408946	0.0736165	0.1124813	0.1536036	0.1894568	0.2177351	0.2446080	0.2700841
0.11	0.8	0.0310357	0.0565628	0.0884193	0.1239508	0.1605987	0.1917031	0.2168311	0.2408585
0.11	0.9	0.0242839	0.0444689	0.0704710	0.1007195	0.1332061	0.1662115	0.1935297	0.2161436
0.11	1.0	0.0194820	0.0356879	0.0569366	0.0824032	0.1107004	0.1403631	0.1702171	0.1943594
0.12	0.7	0.0440533	0.0784652	0.1186394	0.1606400	0.1940469	0.2229395	0.2503331	0.2763889
0.12	0.8	0.0335532	0.0606342	0.0938758	0.1304430	0.1678043	0.1963470	0.2220209	0.2465596
0.12	0.9	0.0263161	0.0478719	0.0752355	0.1066176	0.1399466	0.1735489	0.1982172	0.2213224
0.12	1.0	0.0211454	0.0385343	0.0610521	0.0876768	0.1169079	0.1472704	0.1774170	0.1990668

Table 3-56

RESISTANCE FUNCTIONS q/f'_{dc} FOR BEARING WALLS SUPPORTING FIXED-EDGE, ORTHOTROPIC TWO-WAY REINFORCED CONCRETE SLABS, LONG-SPAN DIRECTION (f_{dy} = 75,000 psi)

q_d		Ratio of Effective Wall Thickness, d in., to Clear Span of Slab, L ft							
Wall	α	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
0.03	0.7	0.0125840	0.0260858	0.0478993	0.0780333	0.1130005	0.1489860	0.1838128	0.2068250
0.03	0.8	0.0091366	0.0184785	0.0333731	0.0552086	0.0831801	0.1145336	0.1466280	0.1778946
0.03	0.9	0.0070204	0.0137891	0.0243651	0.0400596	0.0614248	0.0873762	0.1157833	0.1447514
0.03	1.0	0.0055500	0.0107031	0.0185324	0.0300151	0.0460059	0.0665447	0.0905356	0.1163519
0.04	0.7	0.0164870	0.0332749	0.0584842	0.0908048	0.1266166	0.1627430	0.1929165	0.2139169
0.04	0.8	0.0121105	0.0239562	0.0419779	0.0666138	0.0962708	0.1282915	0.1605107	0.1897763
0.04	0.9	0.0092873	0.0180429	0.0312397	0.0497638	0.0734224	0.1007067	0.1296579	0.1567370
0.04	1.0	0.0073574	0.0140811	0.0240425	0.0380591	0.0565262	0.0789397	0.1040052	0.1302794
0.05	0.7	0.0202676	0.0399791	0.0679716	0.1020604	0.1386683	0.1750714	0.1492982	0.2208848
0.05	0.8	0.0149726	0.0291759	0.0498791	0.0767691	0.1078043	0.1404739	0.1729354	0.1960758
0.05	0.9	0.0115204	0.0221545	0.0376917	0.0585725	0.0840618	0.1124513	0.1419447	0.1712408
0.05	1.0	0.0091448	0.0173760	0.0292983	0.0455087	0.0659951	0.0899034	0.1158758	0.1426135
0.06	0.7	0.0239365	0.0462850	0.0766454	0.1122383	0.1495999	0.1823307	0.2055791	0.2277427
0.06	0.8	0.0177768	0.0341710	0.0572254	0.0860130	0.1182315	0.1515230	0.1815023	0.2022759
0.06	0.9	0.0137211	0.0261373	0.0437905	0.0666956	0.0937208	0.1230694	0.1530891	0.1808597
0.06	1.0	0.0109128	0.0205937	0.0343321	0.0524789	0.0746762	0.0998403	0.1266076	0.1538012
0.07	0.7	0.0275031	0.0522561	0.0846846	0.1215992	0.1596746	0.1879082	0.2117703	0.2345028
0.07	0.8	0.0205263	0.0389482	0.0641195	0.0945540	0.1278205	0.1617064	0.1870638	0.2083875
0.07	0.9	0.0158908	0.0300028	0.0495886	0.0742719	0.1026289	0.1328338	0.1633607	0.1864192
0.07	1.0	0.0126620	0.0237394	0.0391699	0.0590519	0.0827387	0.1089963	0.1364772	0.1641128
0.08	0.7	0.0309756	0.0579408	0.0922110	0.1303130	0.1677840	0.1934134	0.2178813	0.2411754
0.08	0.8	0.0232243	0.0435893	0.0706359	0.1025317	0.1367458	0.1697789	0.1925531	0.2144199
0.08	0.9	0.0180308	0.0337607	0.0551264	0.0813990	0.1109380	0.1419223	0.1714011	0.1918968
0.08	1.0	0.0143929	0.0268177	0.0438329	0.0652886	0.0902983	0.1175254	0.1456640	0.1721380
0.09	0.7	0.0343609	0.0633767	0.0993118	0.1384978	0.1725792	0.1988534	0.2239201	0.2477694
0.09	0.8	0.0258735	0.0480523	0.0768305	0.1100468	0.1451286	0.1746303	0.1979776	0.2203809
0.09	0.9	0.0201422	0.0374195	0.0604361	0.0881484	0.1187548	0.1504585	0.1762982	0.1973096
0.09	1.0	0.0161062	0.0298328	0.0483388	0.0712361	0.0974390	0.1255459	0.1542928	0.1770559
0.10	0.7	0.0376654	0.0685937	0.1060519	0.1462397	0.1773226	0.2042367	0.2298938	0.2542928
0.10	0.8	0.0284766	0.0523726	0.0827468	0.1171659	0.1530574	0.1794293	0.2033433	0.2262777
0.10	0.9	0.0222262	0.0409867	0.0655438	0.0945744	0.1261576	0.1585323	0.1811423	0.2026638
0.10	1.0	0.0178024	0.0327884	0.0527025	0.0769310	0.1042235	0.1331377	0.1618164	0.1819205
0.11	0.7	0.0408946	0.0736165	0.1124813	0.1532477	0.1820188	0.2095626	0.2358086	0.2607522
0.11	0.8	0.0310357	0.0565628	0.0884193	0.1239508	0.1587419	0.1841806	0.2086559	0.2321163
0.11	0.9	0.0242839	0.0444689	0.0704710	0.1007195	0.1332061	0.1631353	0.1859383	0.2079650
0.11	1.0	0.0194820	0.0356879	0.0569366	0.0824032	0.1107004	0.1403631	0.1661510	0.1867368
0.12	0.7	0.0440533	0.0784652	0.1186394	0.1572293	0.1866723	0.2148422	0.2416699	0.2671536
0.12	0.8	0.0335532	0.0606342	0.0938758	0.1304430	0.1628576	0.1888885	0.2139203	0.2379020
0.12	0.9	0.0263161	0.0478719	0.0752355	0.1066176	0.1399466	0.1673580	0.1906906	0.2132180
0.12	1.0	0.0211454	0.0385343	0.0610521	0.0876768	0.1169079	0.1472704	0.1704461	0.1915093

	Walls	Column
$f'_{dc} = 2500$ psi	$q = 237$ psi	$\frac{q}{b} \frac{B}{(\text{column})} = 19.8$ psi
$f'_{dc} = 3750$ psi	$q = 356$ psi	$\frac{q}{b} \frac{B}{(\text{column})} = 29.7$ psi
$f'_{dc} = 5000$ psi	$q = 474$ psi	$\frac{q}{b} \frac{B}{(\text{column})} = 39.5$ psi

Equation 3.35. 6, specifically stated in terms of one-way slab parameters, is expressed as Equation 3.35. 12. This latter equation, when solved repetitively over a wide range of loadings, span and material considerations, yields the following general cost guide lines.

(a) For relatively low equivalent loads ($0 < q < 80$ psi) use $f'_{dc} = 2500$ psi for maximum economy.

(b) For medium to high equivalent loads ($80 < q < 250$ psi) use $f'_{dc} = 3750$ psi for maximum economy.

(c) The total percentage ϕ_t of vertical steel in the column should lie between 0.6 and 1.2 percent. The larger percentage is associated with the higher overpressure and longer spans.

(d) Applying the cost factors of Chapter 2, the use of the higher strength reinforcing rod (ASTM 431 or ASTM 432) will generally give the most economical design. It is frequently advisable to investigate both fdy = 60,000 psi and fdy = 75,000 psi.

The most economical design of walls supporting one-way reinforced slabs will involve a series of trial designs, First, guidelines (a) and (b) suggest an optimum value for the concrete strength. Next, guidelines (c) and (d) can be used to bracket values of q_d within a narrow range. Use of Table 3-41 will facilitate the rapid and economical design of bearing walls supporting one-way slabs. The same general guidelines will apply to the design of walls supporting two-slabs. However, bending criteria due to lateral loads may govern the design of walls perpendicular to the long span direction of the slab.

The total cost of a square foot of a bearing wall or lineal foot of a column can be expressed in generalized form as

$$C_t = C_c + C_s + C_{st} + C_f \qquad (3.35.32)$$

The unit cost of the concrete, C_c, is as follows:

All Walls $$C_c = \left[\frac{D}{12}\right] X_c \qquad (3.35.33a)$$

All Columns $$C_c = \left[\frac{bD}{144}\right] X_c \qquad (3.35.33b)$$

The unit cost of the reinforcing steel is

Axially-Loaded Walls $$C_s = \left[\frac{D}{12}\right]\left[\frac{\phi_t}{100}\right] X_s \qquad (3.35.34a)$$

Axially-Loaded Columns $$C_s = \left[\frac{bD}{144}\right]\left[\frac{\phi_t}{100}\right] X_s \qquad (3.35.34b)$$

Eccentrically-Loaded Walls $$C_s = \left[\frac{d}{12}\right]\left[\frac{\phi'_t}{100}\right] X_s \qquad (3.35.34c)$$

Eccentrically-Loaded Columns $$C_s = \left[\frac{bd}{144}\right]\left[\frac{\phi'_t}{100}\right] X_s \qquad (3.35.34d)$$

The unit cost of temperature reinforcement is

All Walls $$C_{st} = \left[\frac{D}{12}\right]\left[\frac{\phi_{te}}{100}\right] X_s \qquad (3.35.35a)$$

All Columns $$C_{st} = \left[\frac{bD}{144}\right]\left[\frac{\phi_{te}}{100}\right] X_s \qquad (3.35.35b)$$

The unit cost of formwork is

All Walls $$C_f = X_f \qquad (3.35.36a)$$

All Columns $$C_f = \left[\frac{b+D}{6}\right] X_f \qquad (3.35.36b)$$

3.36 Flat Slabs

Flat slab construction is readily applicable to buried shelter construction. Flat slab roof and floor systems act monolithically with exterior load-bearing walls, forming a particularly resistant structure. The interior columns permit relatively clear interior spaces, without the height restrictions which the cross-beams of a framed structural system would impose.

The ultimate flexural resistance of a flat slab can be expressed as[2],

$$q_f = K f_{dy} \phi_k \left(\frac{d}{L_L}\right)^2 \tag{3.36.1}$$

In this equation, the coefficient K is calculated as follows[2]:

$$K = \frac{0.0005}{\left[1 + \left(\frac{L_S}{L_L}\right)^2\right]\left[1 - \left(\frac{D_c}{L_L}\right)^2\right]\left[1 - \frac{3}{2}\frac{D_c}{L_L}\right]} \tag{3.36.2}$$

where

D_c = diameter of circular column capital, (ft)

The term ϕ_k in Equation 3.36.1 is expressed as[2],

$$\phi_k = \phi_{Lc}(\text{average}) + \phi_{Sc}(\text{average}) + \left(1 - \frac{P_p}{L_L}\right)\left[\phi_{Le}(\text{av.}) + \phi_{Se}(\text{av.})\right] + \frac{P_p}{L_L}\left(\frac{d_p}{d}\right)^2\left[2\phi_e(\text{panel average})\right] \tag{3.36.3}$$

where

ϕ_{Lc}(av.), ϕ_{Sc}(av.) = average percentage of bottom steel for slab in long and short directions, respectively

ϕ_{Le}(av.), ϕ_{Se}(av.) = average percentage of top steel for slab in long and short directions, respectively

ϕ_e(panel average) = average percentage of top steel for square two-way drop panel

P_p = width of square drop panel, (ft)

d_p = effective depth of drop panel, (in.)

d = effective depth of slab, (in.)

For a square slab ($L_S/L_L = \alpha = 1.0$), Equations 3.36.2 and 3.36.3 become

$$K = \frac{0.0005}{2\left[1 - \left(\frac{D_c}{L}\right)^2\right]\left[1 - 1.5\,\frac{D_c}{L}\right]} \tag{3.36.4}$$

$$\phi_k = 2\phi_c(\text{av.}) + 2\phi_e(\text{av.})\left[1 - \frac{P_p}{L}\right] + 2\phi_e(\text{panel av.})\left[\frac{P_p}{L}\left(\frac{d_p}{d}\right)^2\right] \tag{3.36.5}$$

Assuming $\phi_c = \phi_e$ for the slab, we can simplify Equation 3.36.5 as follows.

$$\phi_k = 2\phi_c\left[2 - \frac{P_p}{L} + \frac{\phi_e\,\text{panel}}{\phi_c\,\text{slab}}\left(\frac{P_p}{L}\right)\left(\frac{d_p}{d}\right)^2\right] \tag{3.36.6}$$

This suggests that Equation 3.36.1 can be written, for square two-way flat slabs with equal top and bottom steel, in the form

$$q_f = k_f f_{dy} \phi_c \left(\frac{d}{L}\right)^2 \qquad (3.36.7)$$

where k_f, a flexural coefficient, is obtained as follows.

$$k_f = \frac{0.0005\left[2 - \frac{P_p}{L} + \frac{\phi_e \text{ panel}}{\phi_c \text{ slab}}\left(\frac{P_p}{L}\right)\left(\frac{d_p}{d}\right)^2\right]}{\left[1 - \left(\frac{D_c}{L}\right)^2\right]\left[1 - 1.5\frac{D_c}{L}\right]} \qquad (3.36.8)$$

Failure in a shearing mode is of concern in flat slab design, since tests indicate that high levels of shearing stress can occur in the regions immediately adjacent to the column capital or to the drop panel. Either the capital-drop panel interface or the drop panel-slab interface may be critical for shear, depending upon the relative dimensions, and both possibilities should be investigated.

Two alternative methods of investigating shearing mode failures are presented. The first method, from Reference 2, describes the relationships which must be met if q_f and q_v are to be equal. For this condition, the shearing mode resistance at the capital-drop panel interface is given by

$$q_v = \left[\frac{d_p D_c}{L_L L_S - 0.785 D_c^2}\right]\left[76.3 + 0.01035 f'_c\right] \qquad (3.36.9a)$$

or, for $L_L = L_S$,

$$q_v = \left[\frac{d_p D_c}{L^2 - 0.785 D_c^2}\right]\left[76.3 + 0.01035 f'_c\right] \qquad (3.36.9b)$$

A similar expression can be written for the shearing mode resistance at the drop panel-slab interface.

$$q_v = \left[\frac{d_p P_p}{L_S L_L - P^2_p}\right]\left[97.2 + 0.0132\, f'_c\right] \qquad (3.36.10a)$$

or, for $L_L = L_S$,

$$q_v = \left[\frac{d_p P_p}{L^2 - P^2_p}\right]\left[97.2 + 0.0132\, f'_c\right] \qquad (3.36.10b)$$

An alternative method follows the method used in deriving the shear compression, diagonal tension resistance function for one and two-way slabs. The assumption is made that the diagonal tension coefficients for a flat slab can be obtained from those derived for one-way reinforced slabs with similar edge-support conditions. However, certain approximations are introduced in establishing this relationship. First, as was assumed in the study of two-way slabs, an expression relating the diagonal tension resistance of a two-way flat slab to that of a one-way slab will include a 1.33 multiplying factor, in recognition of two-way slab action(2). Next, the diagonal tension coefficient for the one-way slab is adjusted for flat slabs according to the ratio of the perimeter of the capital, or of the circumscribed circle about a drop panel, to the center-to-center distance between the columns supporting the slab. Finally, consideration is given to the ratio of the gross slab area, measured between column center lines, to the same slab area minus the area of capital or drop panel.

Applying these approximations, the applicable value for the diagonal tension coefficient when a shear failure at the capital -drop panel interface is considered, can be expressed as follows for a square flat slab with two-way reinforcement.

$$k_{sc} = (1.765 \times 1.33)\left(\frac{\pi D_c}{L}\right)\left[\frac{1}{1 - 0.785\left(\frac{D_c}{L}\right)^2}\right]$$

or

$$k_{sc} = 7.374\left(\frac{D_c}{L}\right)\left[\frac{1}{1 - 0.785\left(\frac{D_c}{L}\right)^2}\right] \qquad (3.36.11)$$

Considering the same slab, but examining shearing mode resistance at the drop panel-slab interface,

$$k_{sc} = (1.765 \times 1.33)\left(\frac{1.41\,\pi P_p}{L}\right)\left[\frac{1}{1 - 0.785\left(\frac{1.41 P_p}{L}\right)^2}\right]$$

or

$$k_{sc} = 10.397\left(\frac{P_p}{L}\right)\left[\frac{1}{1 - 0.785\left(\frac{1.41 P_p}{L}\right)^2}\right] \qquad (3.36.12)$$

Values of k_{sc}, as obtained from Equations 3.36.11 and 3.36.12 can be substituted into the generalized equation for diagonal tension resistance of any slab. This includes a term for web reinforcing steel, if required.

$$q_{sc} = \frac{k_{sc}}{(2+\theta')}\left(\frac{d}{L}\right)^2 (f'_c \phi_c)^{1/2}\left[1 + 0.00002\,\phi_v f_{dy}\right] \qquad (3.34.34)$$

The value of d_p is substituted for d in Equation 3.34.34 for the diagonal tension resistance of the drop panel-column interface. Equations 3.35.6 and 3.34.34 can be solved simultaneously, obtaining the same result as was found in Section 3.34.4 for the general slab.

$$\phi_v = 50,000 \left[(2+\theta') \frac{k_f}{k_{sc}} \sqrt{\frac{\phi_c}{f'_c} - \frac{1}{f_{dy}}} \right] \qquad (3.34.41)$$

Many design alternatives are possible by varying the ratios of short-span to long-span or the dimensions for the drop panel and capital. For simplicity, it is assumed that capital dimensions are limited to either 0.2 or 0.3 of the center-to-center span between columns[41] and that span lengths in both directions are equal, $L_S/L_L = \alpha = 1.0$.

Table 3-57 supplies calculated values of k_f and k_{sc} for selected combinations of d_p/d, D_c and P_p in square two-way flat slabs. The calculations for k_f assume that ϕ_c (slab) and ϕ_c (drop panel) are equal, thus permitting direct solutions of Equation 3.36.8.

The drop panel, capital and column structural system also may be used as interior supports for one-way slabs if two-way reinforcement is provided in the drop panel and in the slab running between columns in a band equal to the width of the drop panel.

Table 3-57

FLEXURE AND DIAGONAL TENSION COEFFICIENTS FOR SQUARE FLAT SLABS

Case Considered		k_f	k_{sc}
	$P_p = 0.3L$	0.00162	3.421
$d_p = 1.25d$	$P_p = 0.4L$	0.00165	5.050
$D_c = 0.20L$	$P_p = 0.5L$	0.00170	7.175
	$P_p = 0.3L$	0.00178	3.421
$d_p = 1.50d$	$P_p = 0.4L$	0.00187	5.050
$D_c = 0.20L$	$P_p = 0.5L$	0.00196	7.175

The value of k_{sc} (Equation 3.36.11) equals 1.513 for D_c equal to 0.20L.

This latter design method, while entirely compatible with those proposed for one and two-way slabs (Sections 3.33 and 3.34), involves the use and extrapolation of empirical data which were derived for a wholly

different structural arrangement. Until verified by tests and more rigorous analysis, the method should be treated with extreme caution.

The form of the cost equations for the flat slab is identical with that used for the two-way slab. All that is required is the proper substitution of coefficients from Table 3-57. The slab limits, for costing purposes, are lines drawn between column centers. To this is added the incremental cost of the drop panel or capital. Neglecting the incremental cost of forming the drop, the composite cost factor per square foot of drop panel can be expressed as,

$$C_t = \left(\frac{D_p}{D} - 1\right) C_c + \left(\frac{d_p}{d} - 1\right) C_s + \left(\frac{d_p}{d} - 1\right) C_v \qquad (3.36.13)$$

where

D_p = total depth of drop panel, (in.)

The other notations are the same as used for one and two-way slabs in Sections 3.33 and 3.34. For use with one-way slabs, Equation 3.36.9 must be modified to reflect two-way reinforcement costs in the drop panel and the slab band between columns.

The capital for a flat slab system will normally be a minor cost item in comparison with the other components. For estimating costs, the capital is assumed to consist of a 45° frustum of concrete, extending outward from the column. The central core of the capital, whose diameter is assumed to be 0.1 L, is treated as an extension of the column. One percent of the capital volume, excluding the core, is assumed to consist of steel reinforcement.

For capitals:

when

$$D_c = 0.2\,L$$

then

$$C_T = 0.00052\,(X_c + 0.01\,X_s)\,L^3 + 0.0332\,X_f L^2 \qquad (3.36.14a)$$

when

$$D_c = 0.3\,L$$

then

$$C_T = 0.00262\,(X_c + 0.01\,X_s)\,L^3 + 0.0887\,X_f L^2 \qquad (3.36.14b)$$

where

C_T = total cost of the capital, (\$)

3.37 Single-Curvature Compression Members

Single-curvature reinforced concrete shells can be used in the barrel arch, rib arch and cylinder configuration. The assumptions of a uniform radial loading and of adequate lateral restraint to preclude buckling, introduced when formulating design equations for steel shells, are applied with somewhat less assurance to the analysis of reinforced concrete shells. Sufficient shell flexibility must be mobilized, for both materials, to develop the assumed radial pattern of loading. Lacking adequate quantitative data to predict how the two materials may differ in their response to blast loading, however, the simple compression mode is assumed to be equally valid as a failure criterion for steel shells and for reinforced concrete shells.

The design equation for a single-curvature compression member is similar to that expressed in Section 3.32 for an axially-loaded reinforced concrete column. Typical single-curvature compressive members will have section dimensions D inches x b inches and a span length S_L feet. For such members, when spaced a distance of B feet apart and loaded to their ultimate in the compressive mode, the ultimate resistance can be expressed as,

$$\frac{q_c \times 12B \times 12S_L}{2} = 0.85 f'_{dc} D b + \frac{\phi_t}{100} D b f_{dy} \qquad (3.37.1a)$$

This expression reduces to

$$\frac{q_c B S_L}{b D f'_{dc}} = 0.0118 + 0.0139 q_{dt} \qquad (3.37.1b)$$

For a shell, where B = b/12, the expression for ultimate compressive resistance per lineal inch can be written as,

$$\frac{q_c S_L}{D} = 0.142 f'_{dc} + 0.001667 \phi_t f_{dy} \qquad (3.37.1c)$$

A constant total percentage of reinforcing steel, $\phi_t = 0.50$, is proposed for singly-curved shells. Temperature steel will be provided at right-angles to this main reinforcement. Table 3-58 supplies compressive resistance functions $q_c (S_L/D)$ for various combinations of reinforcing steel, dynamic yield stresses f_{dy} and ultimate dynamic concrete compressive stresses f'_{dc}. The calculated values include the assumption that ϕ_t will have a constant value of 0.50.

Table 3-58

RESISTANCE FUNCTIONS FOR SINGLY-CURVED REINFORCED CONCRETE SHELLS

f'_{dc} psi	Resistance Function $\frac{q_c S_L}{D}$ ($\phi_t = 0.50$) f_{dy} = 44,000 psi	f_{dy} = 52,000 psi	f_{dy} = 60,000 psi	f_{dy} = 75,000 psi
2500	392	398	405	418
3750	569	576	583	595
5000	747	753	760	773
6250	924	931	938	950
7500	1102	1108	1115	1128

The optimum relationship between the cost of a concrete and its compressive strength becomes immediately apparent when design alternatives for singly-curved shells are studied. As an illustration, if shell resistance is governed solely by its compressive strength, 7500 psi is obviously the optimum choice from among concrete strengths in the range from 2500 - 7500 psi. The high strength concrete, although it costs approximately 30 percent more than the low strength concrete, has three times its compressive strength. A similar relationship holds true when shells of plate steel and of concrete are compared. For optimum choices of concrete and

steel shells, a steel which is approximately 13 times stronger in compression than a given concrete, will cost approximately 75 times as much. Thus, within the restriction imposed by minimum dimensional limitations, it is concluded that optimum shell costs will result when minimum amounts of the lowest-cost steel are used with maximum concrete strengths.

The cost of single-curvature compression members of reinforced concrete can be expressed as

$$C_t = C_c + C_s + C_{st} + C_f \qquad (3.37.2)$$

where

C_t = factor for composite cost, \$/sq ft for shell and \$/ft for rib

C_c = cost factor for concrete, \$/sq ft of shell surface and \$/ft for ribs

C_s = cost factor of reinforcing steel, \$/sq ft for shell and \$/ft for rib

C_{st} = cost factor for temperature steel, \$/sq ft for shell

C_f = cost factor for form work, \$/sq ft for shell and \$/ft for rib

and

Ribs $$C_c = \left[\frac{bD}{144}\right] X_c \qquad (3.37.3a)$$

Shells $$C_c = \left[\frac{D}{12}\right] X_c \qquad (3.37.3b)$$

Ribs $$C_s = \left[\frac{\phi_t b D}{14,400}\right] X_s \qquad (3.37.4a)$$

Shells $$C_s = \left[\frac{\phi_t D}{1200}\right] X_s \qquad (3.37.4b)$$

Shells $$C_{st} = \left[\frac{\phi_{te} D}{1200}\right] X_s \qquad (3.37.5)$$

Ribs $$C_f = \left[\frac{b+D}{6}\right] X_f \qquad (3.37.6a)$$

Shells $$C_f = X_f \qquad (3.37.6b)$$

In these equations, ϕ_{te} is the total temperature reinforcement expressed as a percentage of the total cross-sectional area of the compressive member. All other terms are as previously defined.

3.38 Double-Curvature Compression Members

Double-curvature compression members occur in domes and spheres. Extending the assumptions of radially-applied load and of constraint of the possible buckling modes, as described for singly-curved compression members, the doubly-curved shells act in direct compression. Due to its two-way action, however, the ultimate compressive strength of a section of a given thickness is twice that available in a single-curvature shell. However, at least a part of this advantage is offset by increased forming costs.

The design equation for a double-curvature member of length S_L and thickness D, loaded to its ultimate in the compressive mode, can be expressed as,

$$\frac{q_c S_L}{D} = 0.284 f'_{dc} + 0.003333 \phi_t f_{dy} \tag{3.38.1}$$

As with the singly-curved shell, a constant percentage of reinforcement steel $\phi_t = 0.50$ will be used in the analyses. Due to the two-way action of the doubly-curved shell, 0.5 percent of reinforcement will be provided in two directions. The resistance functions listed in Table 3-58 can be doubled for double-curvature shells of the same thickness as the single-curvature shells.

By substituting appropriate values of X_c and X_f (see Chapter 2), Equation 3.37.3b and 3.37.6b can be used to determine cost factors C_c and C_f for doubly-curved shells. Temperature steel will not be required. The expression for C_s, due to the increased minimum reinforcement in the two-way shell, now becomes

$$C_s = 2\left(\frac{X_s \phi_t}{100}\right) \frac{D}{12} \tag{3.38.2}$$

3.39 Footings

Footings have several possible applications in the design of buried shelters, and are treated as a separate structural element. Continuous footings can conceivably support load-bearing walls for all shelter configurations considered in this study, while isolated footings can be used to support columns in flat slab or fully-framed construction. The structural loading on all footings, for the three basic shelter configurations considered herein, is taken to be axial and without any transfer of moment to the footing. It is assumed that lateral thrust at the footing level will be transferred through connecting grade beams which thus function as axially-loaded columns. When preparing detailed designs, the passive earth resistance associated with any lateral footing displacement should also be evaluated.

The footing is thus designed for a concentric axial load and, for a specified dynamic soil-bearing capacity, its minimum plan dimensions are immediately known. The required footing depth may be controlled either by flexural stresses, considering the footing projection as a loaded cantilever, or by shearing stresses in the footing adjacent to the wall or column. The analytical procedure for footing design is similar to that described for reinforced concrete slabs, and involves checking critical load conditions for each anticipated mode of failure. It is postulated that footing slabs will be of uniform depth, as opposed to stepped or sloped footings, and that shear reinforcement will not be provided.

Although the behavior of dynamically-loaded footings has received recent attention[42, 43], there still remains a considerable degree of uncertainty when dynamic bearing values must be predicted for a particular soil. It has been suggested[2] that a safe bearing value can be taken as the sum of the surface overpressure plus twice the "normal" static bearing value. Obviously, the design objective is the avoidance of a general shear failure in the soil beneath the loaded footing, while still permitting shearing displacements of magnitudes which would be considered unacceptable by conventional standards. The time-response characteristics of the soil-footing system are also of interest, since the duration of blast loading is relatively short.

For application in this study, dynamic bearing capacities are computed for selected soils with representative values of ϕ and c. Here ϕ = effective angle of internal sliding resistance for the soil, while c = effective cohesion in pounds per square foot of sliding surface. These bearing capacities, which are shown in Table 3-59, are computed from a theoretical solution[44] which assumes a logarithmic spiral failure surface.

(1) Wall Footings

The wall footing receives an axial dynamic load from the bearing wall and distributes this load over an area of soil. The wall load, represented in the analysis as an equivalent static load P expressed in pounds per lineal foot of wall, will be applied to the footing over the width D inches of the bearing wall. Assuming that the wall has been designed to develop its ultimate resisting capacity, the footing load P will be equal to P_{do} if the reinforced-concrete bearing wall is axially-loaded (Section 3.32) or will equal P_{du} for an eccentrically-loaded reinforced concrete wall (Section 3.35). The footing width, L feet, can be determined as the quotient of the equivalent load P in pounds per lineal foot of wall divided by the permissible dynamic bearing capacity of the soil (psf). Main reinforcing steel, ϕ_c, is located in the lower part of the footing and placed perpendicular to the plane of the bearing wall. Minimum compressive reinforcement, $\theta' = 0.25$, is placed in the top of the footing to provide for possible rebound stresses. Finally, temperature reinforcement is placed along the length of the footing, parallel to the bearing wall.

The maximum flexural stresses will occur at the face of the bearing wall where plastic yielding, due to cantilever bending, can develop. The ability of the wall footing to resist diagonal tension stresses is analyzed by assuming the footing to act as a wide beam[1], cantilevered outward from the bearing wall. Finally, its resistance to "pure" shear is checked at a pseudo-critical section[45] at a distance of D/2 from the face of the bearing wall. Design equations[5, 46] for the continuous footing are as follows:

Table 3-59

ULTIMATE DYNAMIC BEARING CAPACITY, kips/ft
FOR CONTINUOUS FOOTINGS OF WIDTH L, ft

(Logarithmic Spiral Solution)

q (psi)	ϕ	c (psf)	L = 4'-0"	L = 6'-0"	L = 8'-0"	L = 10'-0"
25	30°	0	60	119	195	290
	15°	2000	96	147	200	255
	15°	4000	197	298	401	507
	0°	4000	74	111	149	186
50	30°	0	47	98	167	255
	15°	2000	85	130	177	230
	15°	4000	185	281	379	479
	0°	4000	63	94	126	157
75	30°	0	30	74	136	217
	15°	2000	73	119	153	195
	15°	4000	174	265	356	450
	0°	4000	51	76	101	127
100	30°	0	16	46	102	176
	15°	2000	60	93	128	164
	15°	4000	163	247	333	421
	0°	4000	38	57	75	94
125	30°	0	---	---	59	129
	15°	2000	47	73	101	130
	15°	4000	151	229	310	392
	0°	4000	23	34	45	57

Flexural Mode

$$\frac{P}{L} = 0.072\,\phi_c\, f_{dy}\left[\frac{d_{footing}}{L - \frac{D_{wall}}{12}}\right]^2 \qquad (3.39.1a)$$

or

$$\phi_c = \frac{P}{L}\left(\frac{0.09653}{f_{dy}}\right)\left[\frac{12\,L - D_{wall}}{d_{footing}}\right] \qquad (3.39.1b)$$

where

ϕ_c	=	percentage, referenced to net area of section, of tension reinforcing steel at the face of the bearing wall
d	=	the effective depth of footing, (in.)
D	=	the width of bearing wall, (in.)
L	=	total width of continuous footing, (ft)
P	=	total axial load per lineal foot of continuous footing, (lb)

Diagonal Tension Mode

$$\frac{P}{L} = 100 \quad \sqrt{f'_c\,\phi_c}\left[\frac{d_{footing}}{L - \frac{D_{wall}}{12}}\right]^2 \qquad (3.39.2)$$

Shear Mode

$$\frac{P}{L} = \left[\frac{96\, d_{footing}\ \sqrt{f'_c}}{L - \frac{D_{wall}}{12} - \frac{d_{footing}}{12}}\right] \qquad (3.39.3a)$$

or

$$\frac{d_{footing}}{L} = \left[\frac{\left(12 - \frac{D_{wall}}{L}\right)}{\frac{1152 \sqrt{f'_c}}{\frac{P}{L}} + 1} \right] \qquad (3.39.3b)$$

Equations 3.39.1 and 3.39.2 can be solved simultaneously for balanced strengths in flexure and in diagonal tension, following the procedure described in the analyses of one-way and two-way slabs.

$$\phi_c = \frac{1,930,000 f'_c}{f^2_{dy}} \quad \text{when } q_f = q_{sc} \qquad (3.39.4)$$

However, the design of a wall footing will normally be governed by flexural and "pure" shear stresses. It is thus more advantageous to solve Equation 3.39.1 and 3.39.3 simultaneously, obtaining

$$\phi_c = \frac{1334}{d_{footing}} \frac{\sqrt{f'_c}}{f_{dy}} \left[\frac{L - \frac{D_{wall}}{12}}{1 - \left(\frac{d_{footing}}{12L - D_{wall}}\right)} \right] \quad \text{when } q_f = q_v \qquad (3.39.5)$$

The design of a continuous wall footing will involve the following steps:

a) Select the required footing width L based upon known load P pounds per lineal foot and the specified bearing capacity for the soil.

b) For a selected value of f'_c, Equation 3.39.3 is then solved to obtain the required effective depth of footing d, as controlled

by "pure" shearing stresses. Table 3-60 supplies values of d/L for selected ranges of P/L, D_{wall}/L and f'_c.

c) The effective depth of footing d, as controlled by shear, Equation 3.39.3 or Table 3-60, is then substituted into Equation 3.39.1. This identifies the percentage of tensile reinforcement, ϕ_c, which for the effective footing depth, d, and a selected steel dynamic yield-stress, f_{dy}, ensures that the footing will have the required flexural resistance. By computing $\left[\frac{d}{12L - D}\right]$, Table 3-61 can be used to obtain ϕ_c values for specified levels of P/L^2 and f_{dy}.

d) The use of an effective depth of footing d, as obtained from Equation 3.39.3 or Table 3-60, with a percentage of tensile reinforcement, ϕ_c, as obtained from Equation 3.39.5 or Table 3-61, will result in a footing with theoretically equal ultimate resistances in flexure and in "pure" shear. As a final step, the resistance of the footing in diagonal tension must be checked to ensure that this latter mode does not control. If the value of ϕ_c which is required by flexural stresses, Equation 3.39.5, does not exceed values of ϕ_c calculated by Equation 3.39.4, diagonal tension will not be critical. Otherwise, the critical failure modes are identified as "pure" shear and diagonal tension rather than flexure and diagonal tension. For this case, which will rarely occur, a solution must be obtained from Equation 3.39.2 and 3.39.3.

(2) Square Column Footings

Equations are developed only for those two-way reinforced axially-loaded footings which are square in plan. However, if subsequently desired, the same equations could readily be extended to rectangular column footings. The dynamic load transferred to the footing is represented in the analysis by an equivalent static load, P. This column load, which is assumed to be axial, may be applied either directly to the footing or through a base plate. For both cases, the analysis will assume that the load in the footing is applied uniformly over a square area with plan dimensions of D inches. For a column

Table 3-60

MINIMUM RATIOS OF EFFECTIVE FOOTING DEPTH TO FOOTING WIDTH, d/L, (in./ft) TO AVOID SHEAR FAILURE IN CONTINUOUS FOOTING WITH EQUIVALENT UNIT LOAD P/L (lb/ft/ft)

f'_c	P/L	Ratio of Width of Bearing Wall to Footing Width, D/L (in./ft)								
psi	lb/ft/ft	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
2000.	2000.	0.41137	0.39238	0.37370	0.35501	0.33633	0.31764	0.29895	0.28027	0.26157
	4000.	0.79252	0.75650	0.72047	0.68445	0.64943	0.61240	0.57538	0.54035	0.50433
	6000.	1.14745	1.09527	1.04313	0.99078	0.93882	0.88666	0.83451	0.78235	0.73017
	8000.	1.47852	1.41131	1.34411	1.27670	1.20770	1.14249	1.07527	1.00805	0.94088
	10000.	1.72307	1.70579	1.62551	1.54424	1.46296	1.38169	1.30041	1.21914	1.13786
	15000.	2.45049	2.36774	2.25499	2.14224	2.02950	1.91675	1.80400	1.69125	1.57850
	20000.	3.07511	2.93528	2.79646	2.65564	2.51681	2.37699	2.23717	2.09734	1.95752
	25000.	3.59388	3.43052	3.25716	3.10380	2.94045	2.77709	2.61373	2.45037	2.28701
	30000.	4.04314	3.86413	3.68012	3.49512	3.31211	3.12810	2.94410	2.76009	2.57609
	40000.	4.60774	4.58021	4.37068	4.15214	3.93361	3.71508	3.49654	3.27301	3.05947
	50000.	5.41770	5.17145	4.92519	4.67493	4.43267	4.18641	3.94015	3.69389	3.44763
	60000.	5.91827	5.64926	5.38025	5.11124	4.84222	4.57321	4.30420	4.03519	3.76617
2500.	2000.	0.36913	0.35235	0.33557	0.31979	0.30201	0.28523	0.26846	0.25168	0.23490
	4000.	0.71429	0.68182	0.64935	0.61668	0.58442	0.55195	0.51948	0.48701	0.45455
	6000.	1.03774	0.99057	0.94340	0.89623	0.84906	0.80189	0.75472	0.70755	0.66038
	8000.	1.34146	1.28049	1.21951	1.15854	1.09756	1.03659	0.97561	0.91463	0.85366
	10000.	1.62722	1.55325	1.47929	1.40533	1.33136	1.25740	1.18343	1.10947	1.03550
	15000.	2.27273	2.16942	2.06612	1.96281	1.85950	1.75620	1.65289	1.54959	1.44628
	20000.	2.83505	2.70619	2.57732	2.44845	2.31959	2.19072	2.06186	1.93299	1.80412
	25000.	3.32930	3.17797	3.02663	2.87530	2.72397	2.57264	2.42131	2.26998	2.11864
	30000.	3.76712	3.59599	3.42466	3.25342	3.06219	2.91096	2.73973	2.56849	2.39726
	40000.	4.50820	4.30328	4.09836	3.89344	3.68852	3.48361	3.27869	3.07377	2.86885
	50000.	5.11152	4.87918	4.64684	4.41450	4.18216	3.94981	3.71747	3.48513	3.25279
	60000.	5.61224	5.35714	5.10204	4.84694	4.59184	4.33673	4.08163	3.82653	3.57143
3000.	2000.	0.33795	0.32259	0.30723	0.29187	0.27651	0.26115	0.24578	0.23042	0.21506
	4000.	0.65576	0.62595	0.59615	0.56634	0.53653	0.50672	0.47692	0.44711	0.41730
	6000.	0.95517	0.91175	0.86834	0.82492	0.78150	0.73809	0.69467	0.65125	0.60784
	8000.	1.23773	1.18147	1.12521	1.06895	1.01269	0.95643	0.90017	0.84391	0.78765
	10000.	1.50484	1.43643	1.36803	1.29963	1.23123	1.16283	1.09443	1.02602	0.95762
	15000.	2.11274	2.01671	1.92067	1.82464	1.72861	1.63257	1.53654	1.44050	1.34447
	20000.	2.64749	2.52715	2.40681	2.28647	2.16613	2.04579	1.92545	1.80511	1.68477
	25000.	3.12154	2.97965	2.83776	2.69587	2.55398	2.41210	2.27021	2.12832	1.98643
	30000.	3.54467	3.38354	3.22242	3.06130	2.90018	2.73906	2.57794	2.41682	2.25570
	40000.	4.26780	4.07381	3.87982	3.68583	3.49184	3.29784	3.10385	2.90986	2.71587
	50000.	4.86305	4.64201	4.42096	4.19991	3.97886	3.75781	3.53677	3.31572	3.09467
	60000.	5.36160	5.11789	4.87418	4.63047	4.38676	4.14305	3.89934	3.65563	3.41193

Table 3-60 (Continued)

MINIMUM RATIOS OF EFFECTIVE FOOTING DEPTH TO FOOTING WIDTH, d/L, (in./ft) TO AVOID SHEAR FAILURE IN CONTINUOUS FOOTING WITH EQUIVALENT UNIT LOAD P/L (lb/ft/ft)

f'_c psi	P/L lb/ft/ft	Ratio of Width of Bearing Wall to Footing Width, D/L (in./ft) 1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
4000.	2000.	0.29383	0.28053	0.26717	0.25381	0.24045	0.22709	0.21374	0.20038	0.18702
	4000.	0.57248	0.54646	0.52043	0.49441	0.46839	0.44237	0.41635	0.39033	0.36430
	6000.	0.83694	0.79890	0.76085	0.72281	0.68477	0.64672	0.60868	0.57064	0.53260
	8000.	1.08842	1.03885	0.98938	0.93791	0.89044	0.84097	0.79150	0.74203	0.69256
	10000.	1.32756	1.26721	1.20687	1.14653	1.08618	1.02584	0.96550	0.90515	0.84481
	15000.	1.87801	1.79265	1.70728	1.62192	1.53656	1.45119	1.36583	1.28046	1.19510
	20000.	2.36913	2.26150	2.15381	2.04612	1.93843	1.83074	1.72304	1.61535	1.50766
	25000.	2.81017	2.68243	2.55470	2.42696	2.29923	2.17149	2.04376	1.91602	1.78829
	30000.	3.20828	3.06245	2.91662	2.77079	2.62496	2.47912	2.33329	2.18746	2.04163
	40000.	3.89867	3.72146	3.54425	3.36704	3.18982	3.01261	2.83540	2.65817	2.48077
	50000.	4.47668	4.27320	4.06971	3.86622	3.66274	3.45925	3.25577	3.05226	2.84580
	60000.	4.96769	4.74187	4.51607	4.29027	4.06446	3.83866	3.61286	3.38705	3.16175
5000.	2000.	0.26360	0.25162	0.23964	0.22765	0.21568	0.20369	0.19171	0.17973	0.16775
	4000.	0.51487	0.49147	0.46806	0.44466	0.42126	0.39785	0.37445	0.35105	0.32754
	6000.	0.75464	0.72034	0.68604	0.65174	0.61743	0.58313	0.54883	0.51453	0.48023
	8000.	0.98369	0.93898	0.89427	0.84955	0.80484	0.76013	0.71541	0.67073	0.62533
	10000.	1.20273	1.14806	1.09339	1.03872	0.98405	0.92938	0.87471	0.82004	0.76537
	15000.	1.71056	1.63282	1.55507	1.47732	1.39756	1.32181	1.24405	1.16530	1.08355
	20000.	2.16837	2.06981	1.97125	1.87268	1.77412	1.67556	1.57703	1.47843	1.37957
	25000.	2.58316	2.46574	2.34833	2.23091	2.11350	1.99606	1.87865	1.76125	1.64333
	30000.	2.96074	2.82616	2.69158	2.55700	2.42242	2.28784	2.15326	2.01866	1.88411
	40000.	3.62263	3.45797	3.29330	3.12864	2.96397	2.79931	2.63464	2.46996	2.30531
	50000.	4.18382	3.99365	3.80348	3.61330	3.42313	3.23295	3.04278	2.85251	2.66243
	60000.	4.66567	4.45360	4.24152	4.02944	3.81737	3.60529	3.39322	3.18114	2.96705
6000.	2000.	0.24114	0.23016	0.21922	0.20826	0.19730	0.18633	0.17537	0.16441	0.15345
	4000.	0.47193	0.45048	0.42903	0.40758	0.38613	0.36468	0.34322	0.32177	0.30032
	6000.	0.69303	0.66153	0.63003	0.59853	0.56703	0.53553	0.50402	0.47252	0.44102
	8000.	0.90504	0.86390	0.82276	0.78162	0.74047	0.69935	0.65821	0.61707	0.57591
	10000.	1.10850	1.05811	1.00772	0.95734	0.90695	0.85657	0.80618	0.75579	0.70541
	15000.	1.58293	1.51103	1.43908	1.36712	1.29517	1.22321	1.15126	1.07931	1.00735
	20000.	2.01403	1.92249	1.83094	1.73939	1.64785	1.55630	1.46475	1.37320	1.28156
	25000.	2.40735	2.29792	2.18850	2.07907	1.96965	1.86022	1.75060	1.64137	1.53175
	30000.	2.76766	2.64187	2.51607	2.39027	2.26446	2.13866	2.01246	1.88705	1.76125
	40000.	3.40459	3.24993	3.09517	2.94041	2.78565	2.63090	2.47614	2.32138	2.16552
	50000.	3.95020	3.77064	3.59109	3.41153	3.23198	3.05243	2.87287	2.69332	2.51376
	60000.	4.42260	4.22157	4.02054	3.81952	3.61849	3.41746	3.21644	3.01541	2.81435

Table 3-61

MINIMUM REQUIRED PERCENTAGE OF TENSILE REINFORCEMENT, ϕ_c, FOR CONTINUOUS OR SQUARE FOOTINGS WITH EFFECTIVE DEPTH D (in.) AND EQUIVALENT UNIT LOAD P/L OR P/L^2 (lb/ft/ft or lb/sq ft)

f_{dy}	P/L^2	Values of $\left[\frac{d_{footing}}{12L - D_{wall}}\right]$								
psi	lb/ft^2	0.05	0.10	0.15	0.20	0.25	0.30	0.35	0.40	0.45
44000.	2000.	1.755091	0.438773	0.195010	0.109693	0.070204	0.048753	0.035818	0.027423	0.021669
	4000.	3.510182	0.877545	0.390020	0.219386	0.140407	0.097505	0.071636	0.054847	0.043336
	6000.	5.265273	1.316318	0.585030	0.329080	0.210611	0.146258	0.107455	0.082270	0.065003
	8000.	7.020364	1.755091	0.780040	0.438773	0.280815	0.195010	0.143273	0.109693	0.086671
	10000.	8.775454	2.193864	0.975050	0.548466	0.351018	0.243763	0.179091	0.137116	0.108339
	15000.	13.163182	3.290795	1.462576	0.822699	0.526527	0.365644	0.268636	0.205675	0.162508
	20000.	17.550909	4.387727	1.950101	1.096932	0.702036	0.487525	0.358182	0.274233	0.216678
	25000.	21.938636	5.484659	2.437626	1.371165	0.877545	0.609407	0.447727	0.342791	0.270847
	30000.	26.326363	6.581591	2.925151	1.645398	1.053055	0.731288	0.537273	0.411349	0.325017
	40000.	35.101818	8.775454	3.900202	2.193864	1.404073	0.975050	0.716364	0.548465	0.433356
	50000.	43.877271	10.969318	4.875252	2.742329	1.755091	1.218813	0.895455	0.685582	0.541595
	60000.	52.652726	13.163182	5.850303	3.290795	2.106109	1.462576	1.074545	0.822699	0.650034
52000.	2000.	1.485077	0.371269	0.165009	0.092817	0.059403	0.041252	0.030308	0.023204	0.018334
	4000.	2.970154	0.742538	0.330017	0.185635	0.118806	0.082504	0.060615	0.046409	0.036669
	6000.	4.455231	1.113808	0.495026	0.278452	0.178209	0.123756	0.090923	0.069613	0.055003
	8000.	5.940307	1.485077	0.660034	0.371269	0.237612	0.165009	0.121231	0.092817	0.073337
	10000.	7.425384	1.856346	0.825043	0.464087	0.297015	0.206261	0.151538	0.116022	0.091671
	15000.	11.138077	2.784519	1.237564	0.696130	0.445523	0.309391	0.227308	0.174032	0.137507
	20000.	14.850769	3.712592	1.650085	0.928173	0.594031	0.412521	0.303077	0.232043	0.183343
	25000.	18.563461	4.640865	2.062607	1.160216	0.742538	0.515652	0.378846	0.290054	0.229179
	30000.	22.276154	5.569038	2.475128	1.392260	0.891046	0.618782	0.454615	0.348065	0.275014
	40000.	29.701538	7.425384	3.300171	1.856346	1.188062	0.825043	0.606154	0.464087	0.366686
	50000.	37.126923	9.281731	4.125214	2.320433	1.485077	1.031303	0.757692	0.580108	0.458357
	60000.	44.552307	11.138077	4.950256	2.784519	1.782092	1.237564	0.909231	0.696130	0.550028
60000.	2000.	1.287067	0.321767	0.143007	0.080442	0.051483	0.035752	0.026267	0.020110	0.015890
	4000.	2.574133	0.643533	0.286015	0.160883	0.102965	0.071504	0.052533	0.040221	0.031779
	6000.	3.861200	0.965300	0.429022	0.241325	0.154448	0.107256	0.078800	0.060331	0.047669
	8000.	5.148267	1.287067	0.572030	0.321767	0.205931	0.143007	0.105067	0.080442	0.063559
	10000.	6.435333	1.608833	0.715037	0.402208	0.257413	0.178759	0.131333	0.100552	0.079449
	15000.	9.653000	2.413250	1.072556	0.603312	0.386120	0.268139	0.197000	0.150828	0.119173
	20000.	12.870667	3.217667	1.430074	0.804417	0.514827	0.357519	0.262667	0.201104	0.158897
	25000.	16.088333	4.022083	1.787593	1.005521	0.643533	0.446898	0.328333	0.251380	0.198621
	30000.	19.306000	4.826500	2.145111	1.206625	0.772240	0.536278	0.394000	0.301656	0.238346
	40000.	25.741333	6.435333	2.860148	1.608833	1.029653	0.715037	0.525333	0.402208	0.317794
	50000.	32.176666	8.044167	3.575185	2.011042	1.287067	0.893796	0.656667	0.502760	0.397243
	60000.	38.612000	9.653000	4.290222	2.413250	1.544480	1.072556	0.788000	0.603312	0.476691
75000.	2000.	1.029653	0.257413	0.114406	0.064353	0.041186	0.028601	0.021013	0.016088	0.012712
	4000.	2.059307	0.514827	0.228812	0.128707	0.082372	0.057203	0.042027	0.032177	0.025424
	6000.	3.088960	0.772240	0.343218	0.193060	0.123558	0.085804	0.063040	0.048265	0.038135
	8000.	4.118613	1.029653	0.457624	0.257413	0.164745	0.114406	0.084053	0.064353	0.050847
	10000.	5.148267	1.287067	0.572030	0.321767	0.205931	0.143007	0.105067	0.080442	0.063559
	15000.	7.722400	1.930600	0.858044	0.482650	0.308896	0.214511	0.157600	0.120662	0.095338
	20000.	10.296533	2.574133	1.144059	0.643533	0.411861	0.286015	0.210133	0.160883	0.127118
	25000.	12.870667	3.217667	1.430074	0.804417	0.514827	0.357519	0.262667	0.201104	0.158897
	30000.	15.444800	3.861200	1.716089	0.965300	0.617792	0.429022	0.315200	0.241325	0.190677
	40000.	20.593066	5.148267	2.288118	1.287067	0.823723	0.572030	0.420267	0.321767	0.254235
	50000.	25.741333	6.435333	2.860148	1.608833	1.029653	0.715037	0.525333	0.402208	0.317794
	60000.	30.889600	7.722400	3.432178	1.930600	1.235584	0.858044	0.630400	0.482650	0.381353

which is loaded to its maximum capacity, as described in earlier sections of this report, the column load P is equal to P_{dy} or to P_{do} for axially-loaded columns of structural steel or reinforced concrete, respectively. If these same columns are eccentrically loaded, then P'_{dy} will replace P_{dy} and P_{du} will replace P_{do}

Knowing the equivalent load P and the specified dynamic soil bearing capacity, the required plan dimension L for the square footing can immediately be computed. As was normally the case with the continuous footing, the required depth for a square footing will be controlled by flexural stresses or by shearing stresses. The maximum flexural stresses occur at the face of the column or column base plate where plastic yielding, due to cantilever bending, can develop. Shearing stresses are examined at a pseudo-critical section[45, 46], which is a distance of $\frac{D}{2}$ from the face of the column or base plate. No separate analysis is made of diagonal tension resistance, since this mode is not considered to be critical in a square column footing. Equal percentages, ϕ_c, of bottom reinforcement are provided in each direction, while minimum percentages of compressive reinforcement, $\theta' = 0.25$, are placed in the upper part of the footing. No additional reinforcement is considered necessary for temperature stresses, since the footing is two-way reinforced. Design equations are as follows:

Flexural Mode

$$\frac{P}{L^2} = 0.072\,\phi_c f_{dy} \left[\frac{d_{footing}}{L - \frac{D_{wall}}{12}}\right]^2 \qquad (3.39.6a)$$

or

$$\phi_c = \frac{P}{L^2}\left(\frac{0.09653}{f_{dy}}\right)\left[\frac{12L - D_{wall}}{d_{footing}}\right]^2 \qquad (3.39.6b)$$

Equations 3.39.6a and 3.39.6b are seen to be identical with Equations 3.39.1a and 3.39.1b for the continuous footing. The quotient of load per lineal foot and footing width, $\frac{P}{L}$, for the continuous footing is numerically equal to the quotient of total load and footing dimension, P/L^2, for a square footing. Thus, Table 3-61 can also be used to determine ϕ_c for square footings.

Shear Mode

$$\frac{d_{footing}}{D_{wall}} = 0.5\left(\frac{k+2}{k+1}\right)\left[\sqrt{\left(\frac{4}{k+2}\right)\left(\frac{k+1}{k+2}\right)\left(\frac{144L^2}{D^2_{wall}} - 1\right) + 1} - 1\right] \quad (3.39.7)$$

where

$$k = \frac{2310\sqrt{f'_c}}{\frac{P}{L^2}}$$

The design of a square two-way reinforced concrete footing involves the following steps:

a) Select the footing dimension L, based upon known total load P and specified bearing capacity of the soil.

b) After specifying f'_c, Equation 3.39.7 is solved to obtain the required effective depth of footing, d, as controlled by shearing stresses. Table 3-62 supplies values of d/P for selected ranges of P/L^2, f'_c and D_{wall}/L. For this usage, D_{wall} is the width of column or base plate.

c) After selecting the effective depth of footing d, Equation 3.39.6 or Table 3-62 is used to determine the required percentage of tensile reinforcement, ϕ_c, in one direction of the footing.

Table 3-62

MINIMUM RATIOS OF EFFECTIVE FOOTING DEPTH TO WIDTH OF SQUARE COLUMN OR BASE PLATE, d/D (in./in.) TO AVOID SHEAR FAILURE IN ISOLATED SQUARE FOOTING WITH EQUIVALENT UNIT LOAD P/L^2 (lb/sq ft)

f'_c psi	P/L^2 lb/ft²	Ratio of Width of Column or Base Plate to Footing Width, D/L, (in./ft.) 1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
2000.	2000.	1.21546	0.69719	0.45192	0.31402	0.22839	0.17157	0.13201	0.10341	0.08210
	4000.	1.84769	1.09920	0.73582	0.52566	0.39138	0.29983	0.23449	0.18619	0.14951
	6000.	2.32347	1.40552	0.95559	0.69239	0.52211	0.40452	0.31951	0.25591	0.20705
	8000.	2.71447	1.65844	1.13822	0.83203	0.63254	0.49374	0.39262	0.31639	0.25736
	10000.	3.04986	1.87584	1.29572	0.95295	0.72863	0.57179	0.45693	0.36988	0.30214
	15000.	3.73175	2.31847	1.61721	1.20065	0.92632	0.73316	0.59061	0.48171	0.39526
	20000.	4.26987	2.66792	1.87137	1.39691	1.08343	0.86186	0.69767	0.57167	0.47234
	25000.	4.71482	2.95676	2.08148	1.55927	1.21355	0.96862	0.78666	0.64662	0.53588
	30000.	5.09332	3.20232	2.26007	1.69728	1.32419	1.05947	0.86245	0.71053	0.59014
	40000.	5.71052	3.60234	2.55079	1.92186	1.50424	1.20734	0.98587	0.81467	0.67854
	50000.	6.17823	3.91806	2.78000	2.09879	1.64600	1.32373	1.08301	0.89666	0.74835
	60000.	6.59687	4.17573	2.96687	2.24292	1.76141	1.41843	1.16202	0.96333	0.80502
2500.	2000.	1.13241	0.64515	0.41579	0.28756	0.20836	0.15606	0.11978	0.09365	0.07425
	4000.	1.73127	1.02463	0.68267	0.48564	0.36025	0.27511	0.21456	0.16927	0.13622
	6000.	2.18400	1.31553	0.89083	0.64308	0.48330	0.37332	0.29407	0.23497	0.18971
	8000.	2.55737	1.55673	1.05468	0.77571	0.58792	0.45762	0.36296	0.29181	0.23588
	10000.	2.37862	1.76460	1.21523	0.89110	0.67944	0.53180	0.42394	0.34241	0.27913
	15000.	3.53491	2.19065	1.52430	1.12898	0.86903	0.68631	0.55172	0.44911	0.36876
	20000.	4.05613	2.52912	1.77040	1.31897	1.02096	0.81065	0.65504	0.53582	0.44199
	25000.	4.48959	2.81057	1.97514	1.47709	1.14767	0.91456	0.74158	0.60864	0.50367
	30000.	4.86026	3.05114	2.15013	1.61232	1.25607	1.00354	0.81578	0.67117	0.55572
	40000.	5.46890	3.44581	2.43706	1.83403	1.43383	1.14951	0.93760	0.77393	0.64402
	50000.	5.95396	3.75996	2.65525	2.01024	1.57506	1.26549	1.03441	0.85564	0.71347
	60000.	6.35324	4.01826	2.85269	2.15487	1.69092	1.36059	1.11377	0.92262	0.77042
3000.	2000.	1.06801	0.60497	0.38804	0.26734	0.19312	0.14430	0.11055	0.08631	0.06834
	4000.	1.64058	0.96665	0.64146	0.45472	0.33629	0.25614	0.19933	0.15761	0.12511
	6000.	2.07495	1.24526	0.84036	0.60475	0.45321	0.34919	0.27445	0.21887	0.17540
	8000.	2.43416	1.47705	1.00716	0.73174	0.55316	0.42954	0.33996	0.27279	0.22106
	10000.	2.74397	1.67754	1.15205	0.84263	0.64095	0.50056	0.39823	0.32104	0.26126
	15000.	3.37921	2.03956	1.45086	1.07237	0.82383	0.64939	0.52112	0.42349	0.34718
	20000.	3.38618	2.41875	1.69013	1.25693	0.97135	0.77001	0.62124	0.50741	0.41797
	25000.	4.30963	2.69374	1.89015	1.41142	1.09505	0.87139	0.70561	0.57836	0.47800
	30000.	4.67323	2.92977	2.06185	1.54410	1.20138	0.95864	0.77833	0.63760	0.52993
	40000.	5.27345	3.31912	2.34498	1.76289	1.37680	1.10268	0.89851	0.74095	0.61598
	50000.	5.75495	3.63111	2.57168	1.93800	1.51717	1.21796	0.99473	0.82215	0.68500
	60000.	6.15357	3.88913	2.75900	2.08259	1.63303	1.31308	1.07413	0.88916	0.74191

Table 3-62 (Continued)

MINIMUM RATIOS OF EFFECTIVE FOOTING DEPTH
TO WIDTH OF SQUARE COLUMN OR BASE PLATE, d/D (in./in.)
TO AVOID SHEAR FAILURE IN ISOLATED SQUARE FOOTING
WITH EQUIVALENT UNIT LOAD P/L^2 (lb/sq ft)

f'_c psi	P/L^2 lb/ft²	Ratio of Width of Column or Base Plate to Footing Width, D/L, (in./ft) 1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0
4000.	2000.	0.97242	0.54567	0.34733	0.23786	0.17101	0.12732	0.09726	0.07575	0.05969
	4000.	1.50518	0.68038	0.58040	0.40909	0.30107	0.22838	0.17711	0.13964	0.11140
	6000.	1.91158	1.14021	0.76511	0.54778	0.40863	0.31357	0.24559	0.19525	0.15595
	8000.	2.24902	1.35747	0.92099	0.66603	0.50135	0.38782	0.30588	0.24469	0.19775
	10000.	2.54108	1.54619	1.05707	0.76989	0.58331	0.45390	0.35991	0.28928	0.23476
	15000.	3.14311	1.93633	1.33961	0.98671	0.75552	0.59368	0.47502	0.38497	0.31480
	20000.	3.62700	2.25044	1.56776	1.16249	0.89582	0.70821	0.56989	0.46433	0.38160
	25000.	4.03378	2.51451	1.75985	1.31076	1.01443	0.80531	0.65059	0.53208	0.43882
	30000.	4.38513	2.74275	1.92581	1.43897	1.11712	0.88950	0.72070	0.59105	0.48876
	40000.	4.96977	3.12218	2.20179	1.65225	1.28809	1.02983	0.83771	0.68766	0.57242
	50000.	5.44336	3.42926	2.42503	1.82474	1.42638	1.14340	0.93249	0.76963	0.64035
	60000.	5.83885	3.68544	2.61114	1.96847	1.54159	1.23801	1.01147	0.83628	0.69731
5000.	2000.	0.90315	0.50298	0.31824	0.21692	0.15541	0.11539	0.08797	0.06841	0.05401
	4000.	1.40645	0.81771	0.53624	0.37626	0.27586	0.20859	0.16134	0.12603	0.10113
	6000.	1.79198	1.06349	0.71034	0.50646	0.37643	0.28795	0.22490	0.17638	0.14310
	8000.	2.11306	1.25980	0.85798	0.61812	0.46369	0.35759	0.28127	0.22446	0.18162
	10000.	2.39165	1.44957	0.98734	0.71661	0.54121	0.41991	0.33206	0.26628	0.21565
	15000.	2.96812	1.82292	1.25728	0.92340	0.70512	0.55267	0.44115	0.35673	0.27112
	20000.	3.43379	2.12500	1.47560	1.09220	0.83966	0.66232	0.53183	0.43245	0.35472
	25000.	3.82706	2.38036	1.66222	1.23538	0.95410	0.75589	0.60950	0.47756	0.40365
	30000.	4.16818	2.60189	1.82333	1.35780	1.05370	0.83749	0.67738	0.55460	0.45768
	40000.	4.73906	2.97249	2.09292	1.56611	1.22063	0.97444	0.79151	0.65071	0.53733
	50000.	5.20478	3.27460	2.31262	1.73788	1.35675	1.08621	0.88476	0.72735	0.60515
	60000.	5.59615	3.52826	2.49697	1.88030	1.47092	1.17998	0.96303	0.79540	0.66220
6000.	2000.	0.84956	0.47015	0.29600	0.20100	0.14360	0.10639	0.08098	0.06270	0.04951
	4000.	1.32969	0.76915	0.50216	0.35103	0.25656	0.19350	0.14936	0.11730	0.09333
	6000.	1.69873	1.00380	0.66786	0.47451	0.35162	0.26827	0.20906	0.16551	0.13255
	8000.	2.00679	1.20140	0.80891	0.58091	0.43453	0.33425	0.26233	0.20874	0.16822
	10000.	2.27461	1.37398	0.93288	0.67509	0.50848	0.39355	0.31055	0.24853	0.20093
	15000.	2.83041	1.73355	1.19260	0.87373	0.66564	0.52059	0.41471	0.33474	[illegible]
	20000.	3.28113	2.02590	1.40462	1.03675	0.79541	0.62620	0.50192	0.40743	0.33367
	25000.	3.66309	2.27388	1.58479	1.17564	0.90632	0.71680	0.57702	0.47031	0.38664
	30000.	3.99548	2.43974	1.74176	1.29679	1.00325	0.79614	0.64297	0.52557	0.43341
	40000.	4.55418	2.95230	2.00564	1.50066	1.16657	0.93007	0.75451	0.61753	0.51270
	50000.	5.01246	3.14988	2.22193	1.66781	1.30056	1.04007	0.84626	0.69687	0.57854
	60000.	5.39948	3.40082	2.40437	1.80877	1.41358	1.13285	0.92372	0.76222	0.63466

Practical applications of continuous or isolated footings in buried-shelter design are limited by physical conditions. The equivalent load P, which the footing is designed to support, is related to the design level of overpressure and to the structural layout of the shelter. Thus, for a specified bearing capacity of soil and a particular structural system, the feasible use of footings is restricted to a finite range of span lengths and loading pressures, Figure 3-5. In actual practice, after comparative costs have been analyzed, the range of usefulness for footings may be still further limited. The design alternatives to a footing foundation include the use of a raft foundation or, in the case of arch and dome configurations, the substitution of the "full-round" configurations of cylinder and sphere. It will be desirable to analyze each possible solution separately for a proposed shelter, since the parameters of soil strength will vary with the shelter location. However, by assuming a soil of constant strength properties in these analyses, general conclusions are reached as to the relative suitabilities of footings, continuous raft foundations, and "full-round" construction. These relationships will be discussed as part of the cost studies supplied in Chapter 4.

The cost of footings is based on the value of d and ϕ_c as determined from Tables 3-60 to 3-62, inclusive. The values of P/L or P/L^2, and D/L are normally fixed by allowable bearing pressure and by wall or column thickness. The use of high strength concrete and steel (f'_c = 6000 psi and f_{dy} = 75,000 psi) will normally lead to the most economical footing design except in the cases where minimum dimension requirements govern. The use of these values make possible the rapid designs of an economical footing.

The unit cost of footings can be expressed in the general form,

$$C_t = C_c + C_s + C_{st} + C_f \qquad (3.39.8)$$

where

Column or Wall Footing $\qquad C_c = \left[\frac{D}{12}\right] X_c$

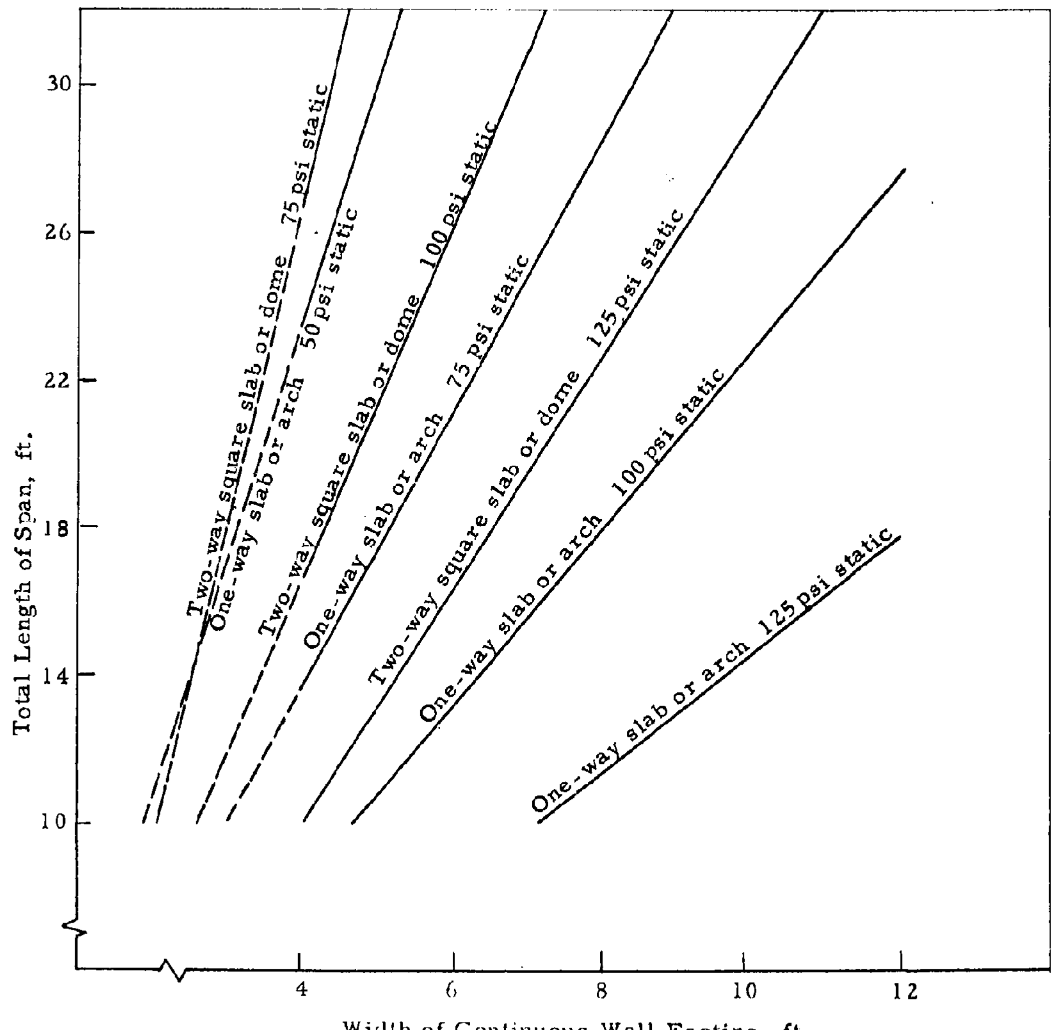


Figure 3-5

RELATION BETWEEN MAXIMUM SPAN OF STRUCTURE AND WIDTH OF CONTINUOUS FOOTING FOR SOIL (ϕ = 15°, c = 2000 psf) STATICALLY LOADED TO ULTIMATE CAPACITY

Wall Footings $$C_s = \frac{d}{12}\left[\frac{\phi_c}{100}\right]X_s$$

Column Footings $$C_s = \frac{d}{6}\left[\frac{\phi_c}{100}\right]X_s$$

Wall Footings $$C_{st} = \frac{D}{12}\left[\frac{\phi_{ts}}{100}\right]X_s$$

Column or
Wall Footing $$C_f = X_f$$

The total cost of a footing, C_T, may be found by multiplying the unit cost C_t by the area (sq ft) covered by the footing.

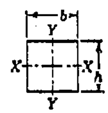
3.4 Structural Timber

3.41 Introduction

Structural timber has several possible applications in buried shelter construction, both in an all-timber design or in composite use with other structural materials. The subsequent analyses will examine the behavior of commercially-available sizes of timber posts, beams and planks. The strength properties of structural timber depends not only upon the material itself, but also upon its environmental conditions of use[20]. While representative strength values are selected for the analytical studies in Chapters 3 and 4, these are not directly applicable to all design situations. It is anticipated that the proposed strength values will be reviewed prior to the preparation of actual timber designs and, if deemed necessary, adjusted to reflect the anticipated service conditions.

The analyses of structural timber elements are based upon elastic behavior and yield point stresses for commercial species, since there is no valid basis for assuming that plastic yielding and internal stress distribution will occur in timber members. However, the static yield stresses are modified in recognition of the behavior of timber under short-duration loading. By removing the customary provision for a factor of safety, and by accepting some increased variability in rated stresses, a factor of 4.0 is derived (see Chapter 2) which may be used to convert conventional working stresses to projected dynamic yield stresses under equivalent static loading. Structural members of a specified nominal size are identified by their commercial species and by their stress grading, hence the design equations are expressed in these terms. Controlling modes of failure, depending on the specific use of a member, may be either flexure, horizontal or vertical shear, compression parallel to the grain, or compression perpendicular to the grain. With the proper selection of stresses and controlling failure modes, the design equations are applicable both to stress-grade lumber and to structural glued-laminated timber. Table 3-63 lists the geometric properties for standard sizes of structural timber.

Table 3-63



GEOMETRIC PROPERTIES FOR STANDARD SIZES OF STRUCTURAL TIMBER

Nominal Size, in Inches	American Standard Dressed Size (S4S), in Inches	Area of Section, in Inches²	Moment of Inertia, in Inches⁴		Section Modulus, in Inches³	
			$I_{X-X} = bh^3/12$	$I_{Y-Y} = b^3h/12$	$S_{X-X} = bh^2/6$	$S_{Y-Y} = b^2h/6$
b h	b h	$A = b \times h$				
2 x 4	1⅝ x 3⅝	5.89	6.45	1.30	3.56	1.60
2 x 6	1⅝ x 5⅝	9.14	24.10	2.01	8.57	2.48
2 x 8	1⅝ x 7½	12.19	57.13	2.68	15.23	3.30
2 x 10	1⅝ x 9½	15.44	116.10	3.40	24.44	4.18
2 x 12	1⅝ x 11½	18.69	205.95	4.11	35.82	5.06
2 x 14	1⅝ x 13½	21.94	333.18	4.83	49.36	5.94
2 x 16	1⅝ x 15½	25.19	504.27	5.54	65.07	6.82
2 x 18	1⅝ x 17½	28.44	725.73	6.25	82.94	7.70
3 x 4	2⅝ x 3⅝	9.52	10.42	5.46	5.75	4.16
3 x 6	2⅝ x 5⅝	14.77	38.93	8.48	13.84	6.46
3 x 8	2⅝ x 7½	19.69	92.29	11.30	24.61	8.61
3 x 10	2⅝ x 9½	24.94	187.55	14.32	39.48	10.91
3 x 12	2⅝ x 11½	30.19	332.69	17.33	57.86	13.21
3 x 14	2⅝ x 13½	35.44	538.21	20.35	79.73	15.50
3 x 16	2⅝ x 15½	40.69	814.60	23.36	105.11	17.80
3 x 18	2⅝ x 17½	45.94	1,172.36	26.38	133.98	20.10
4 x 4	3⅝ x 3⅝	13.14	14.39	14.39	7.94	7.94
4 x 6	3⅝ x 5⅝	20.39	53.76	22.33	19.13	12.32
4 x 8	3⅝ x 7½	27.19	127.44	29.77	33.98	16.43
4 x 10	3⅝ x 9½	34.44	259.00	37.71	54.53	20.81
4 x 12	3⅝ x 11½	41.69	459.43	45.65	79.90	25.19
4 x 14	3⅝ x 13½	48.94	743.24	53.59	110.11	29.57
4 x 16	3⅝ x 15½	56.19	1,124.92	61.53	145.15	33.95
4 x 18	3⅝ x 17½	63.44	1,618.98	69.47	185.03	38.33
6 x 6	5½ x 5½	30.25	76.26	76.26	27.73	27.73
6 x 8	5½ x 7½	41.25	193.36	103.98	51.56	37.81
6 x 10	5½ x 9½	52.25	392.96	131.71	82.73	47.90
6 x 12	5½ x 11½	63.25	697.07	159.44	121.23	57.98
6 x 14	5½ x 13½	74.25	1,127.67	187.17	167.06	68.06
6 x 16	5½ x 15½	85.25	1,706.78	214.90	220.23	78.15
6 x 18	5½ x 17½	96.25	2,456.38	242.63	280.73	88.23
8 x 8	7½ x 7½	56.25	263.67	263.67	70.31	70.31
8 x 10	7½ x 9½	71.25	535.86	333.98	112.81	89.06
8 x 12	7½ x 11½	86.25	950.55	404.30	165.31	107.81
8 x 14	7½ x 13½	101.25	1,537.73	474.61	227.81	126.56
8 x 16	7½ x 15½	116.25	2,327.42	544.92	300.31	145.31
8 x 18	7½ x 17½	131.25	3,349.61	615.23	382.81	164.06
10 x 10	9½ x 9½	90.25	678.76	678.76	142.90	142.90
10 x 12	9½ x 11½	109.25	1,204.03	821.65	209.40	172.98
10 x 14	9½ x 13½	128.25	1,947.80	964.55	288.56	203.06
10 x 16	9½ x 15½	147.25	2,948.07	1,107.44	380.40	233.15
10 x 18	9½ x 17½	166.25	4,242.84	1,250.34	484.90	263.23
12 x 12	11½ x 11½	132.25	1,457.51	1,457.51	253.48	253.48
12 x 14	11½ x 13½	155.25	2,357.86	1,710.98	349.31	297.56
12 x 16	11½ x 15½	178.25	3,568.71	1,964.46	460.48	341.65
12 x 18	11½ x 17½	201.25	5,136.07	2,217.94	586.98	385.73
14 x 14	13½ x 13½	182.25	2,767.92	2,767.92	410.06	410.06
14 x 16	13½ x 15½	209.25	4,189.36	3,177.98	540.56	470.81
14 x 18	13½ x 17½	236.25	6,029.30	3,588.05	689.06	531.56
14 x 20	13½ x 19½	263.25	8,341.73	3,998.11	855.56	592.31

3.42 Axially-Loaded Timber Posts

It is assumed that any timber posts used in buried shelter construction will be axially loaded and will have dimensions such that the ratio of unbraced column height H (ft.) to its least lateral dimensions D (inches) does not exceed unity.

For such short columns, the yield-load capacity is governed by the area of the column and the compressive yield strength of the timber when loaded parallel to its grain. Thus, the maximum column load capacity associated with static loading and conventional yield point stresses is

$$P_y = Af'_{pp} \quad \text{for} \quad \frac{H}{D} \leq 1.0 \qquad (3.42.1a)$$

By recognizing the effective increase in strength which will result from the rapid application of loading, Equation 3.42.1a may be rewritten in terms of dynamic yield stresses as,

$$P_{dy} = Af'_{dpp} \qquad (3.42.1b)$$

The dynamic yield stresses for timber, for the types of use considered in this study, are assumed equal to four times the conventional working stresses for timber, as listed in standard stress grading codes for structural timber[17, 18, 19]. Thus, Equation 3.42.1b may also be written as

$$P_{dy} = 4\,Af_{pp} \qquad (3.42.1c)$$

where

P_y = static yield resistance of an axially-loaded compression member, (lb.)

P_{dy} = dynamic yield resistance of an axially-loaded compression member, (lb.)

A = cross-sectional area of post, (sq. in.)

f'_{pp} = static yield stress for compressive loading of timber parallel to the grain, (psi)

f'_{dpp} = dynamic yield stress for compressive loading of timber parallel to the grain, (psi)

f_{pp} = conventional working stress for static compressive loading of timber parallel to the grain, (psi)

Table 3- 64 contains values of P_{dy} for standard sizes of timber posts, calculated from Equation 3. 42. 1c for representative levels of conventional working stress. In order to make use of this and subsequent tables in this section, it is first necessary to select a species of structural timber. The conventional working stress will then be identified for the selected species and proposed type of loading, using standard reference sources[17, 18, 19] and making appropriate adjustments for any unusual conditions of exposure or service. The values of working stress thus obtained may then be used to enter the tables and obtain resistance functions. These functions incorporate increases in conventional timber stresses, as described in Chapter 2. If use is to be made of a species whose range of working stresses is not included in the tables, the conventional allowable working stress for that species may then be multiplied by 4. 0 and the results used directly for timber shelter design.

If a timber post supports one end of a beam of length L ft. and center-to-center spacing B ft., loaded with an equivalent static load of q_c psi., the yield capacity of the timber compressive member can be expressed as

$$q_c \left(\frac{12L}{2} \times 12B\right) = P_{dy} = Af'_{dpp}$$

or

$$q_c = 0.0139\ Af'_{dpp} \approx 0.0555 Af_{pp} \qquad (3.42.2)$$

3.43 Beams

The yield capacity of a timber beam may be controlled either by flexural stresses or by horizontal shear. As was found to be the case for the steel beam in Section 3. 23, the controlling mode will be determined by the beam length. Horizontal shear will limit the capacity of a short beam,

Table 3-64

DYNAMIC YIELD-LOAD CAPACITIES FOR AXIALLY-LOADED SHORT TIMBER POSTS, P_{dy}, (kips)

NOMINAL SIZE OF POST, in.	CONVENTIONAL WORKING STRESS, psi, FOR COMPRESSION PERPENDICULAR TO THE GRAIN				
	1000	1250	1500	1750	2000
8 x 8	226	283	339	396	452
8 x 10	286	358	430	501	573
8 x 12	347	433	520	607	693
8 x 14	407	509	611	712	814
8 x 16	467	584	701	818	935
8 x 18	528	660	791	923	1055
10 x 10	363	454	544	635	726
10 x 12	439	549	659	769	878
10 x 14	516	644	773	902	1031
10 x 16	592	740	888	1036	1184
10 x 18	668	835	1002	1170	1337
12 x 12	532	665	797	930	1063
12 x 14	624	780	936	1092	1248
12 x 16	717	896	1075	1254	1433
12 x 18	809	1011	1214	1416	1618
14 x 14	733	916	1099	1282	1465
14 x 16	841	1051	1262	1472	1682
14 x 18	950	1087	1425	1662	1899
14 x 20	1058	1323	1587	1852	2117

with flexural stresses remaining below the flexural yield point. At some increased length, assuming that the total load remains constant, the beam will have simultaneously reached its yield strength in flexure and in horizontal shear. This length was designated as L_{fv} in the discussion of the steel beam in Section 3.23 and the same notation will be adopted for the timber beam. However, it should be recognized that L_{fv} for the steel beam was calculated with assumptions of plastification and internal stress redistribution at critical sections, while the calculation of L_{fv} for the timber beam assumes elastic behavior and yield-point stresses. Actually, L_{fv} as calculated for the timber beam is more nearly comparable to L_{ep} for the steel beam.

For beam lengths greater than L_{fv}, the load capacity of the beam is controlled by flexural stresses. Thus, as was found to be the case for the symmetrically-loaded steel beam, total load capacity remains constant for lengths less than L_{fv} and decreases at greater lengths. In the design detailing, for either case, it will also be necessary to ensure that sufficient bearing area is provided at the ends of a beam so that the dynamic yield stress in compression perpendicular to the grain (f'_{dpr}) is not exceeded.

(1) Simply-Supported Beams

For the elastic bending range, the conventional flexural working stress for a timber beam can be related to the external bending moment and to the properties of the section. Since elastic bending is assumed, this level of the working stress will occur on the extreme fibers of the beam

$$f_f = \frac{M}{S} \qquad (3.43.1a)$$

By incorporating the effective increase in flexural yield stress which is postulated in this study, Equation 3.43.1a may be written in terms of the dynamic yield stress at the extreme fibers of the beam

$$f'_{df} = \frac{M}{S} \qquad (3.43.1b)$$

where

f_f = conventional working stress of timber beam in flexure, (psi)

f'_{df} = dynamic yield stress in flexure, (psi)

M = applied bending moment, (in.-lb)

S = section modulus of beam, (in.^{4})

For a simply-supported beam of length L ft. and center-to-center spacing B ft., uniformly-loaded with an equivalent static load of q_f psi, the maximum bending moment occurs at the center of the beam. For this case, the maximum elastic bending moment corresponding to dynamic yield stresses in the extreme fibers of the beam can be written as

$$M = \frac{q_f \times 12B \times 144L^2}{8} = 216\, q_f\, B\, L^2$$

or, substituting $M = S f'_{df}$ and rearranging terms,

$$\frac{q_f\, B\, L^2}{f'_{df}} = \frac{S}{216} \qquad (3.43.2)$$

For a rectangular section of width b in. and total depth D in., $S = \frac{bD^2}{6}$. Thus, by further rearrangement of terms, Equation 3.43.2 becomes

$$\frac{q_f B\, L^2}{f'_{df}} = \frac{b\, D^2}{1296} \qquad (3.43.3)$$

The conventional working stress in horizontal timber shear is related to the total vertical shear by the equation,

$$f_{vh} = \frac{V\, Q}{I\, b} \qquad (3.43.4)$$

where

f_{vh} = conventional working stress for timber in horizontal shear, (psi)

V = vertical shear at the section of the beam being considered, (lb)

Q = statical moment of the cross-sectional area of the section above or below the neutral axis, (in.^{3})

I = moment of inertia of the cross-section, (in.^{4})

b = width of beam, (in.)

For a rectangular section, Equation 3.43.4 reduces to

$$f_{vh} = 1.5 \frac{V}{b\,D} \tag{3.43.5}$$

The maximum shear in a simply-supported beam occurs at the supports. Thus, if a rectangular beam of length L ft. and spacing B ft. is uniformly-loaded with an equivalent static load of q_v psi, Equation 3.43.5 becomes

$$\frac{q_v\,B\,L}{f_{vh}} = \frac{b\,D}{108} \tag{3.43.6}$$

Finally, if the loading is such that the beam is at its dynamic yield stress f'_{dvh} in horizontal shear, this expression may be written as

$$\frac{q_v\,B\,L}{f'_{dvh}} = \frac{b\,D}{108} \tag{3.43.7}$$

Solving Equations 3.43.3 and 3.43.7 simultaneously, we obtain the length of fully-loaded rectangular timber beam ($L = L_{fv}$) at which dynamic yield stresses are simultaneously developed in flexure and in horizontal shear.

$$L_{fv} = \frac{D}{12} \left[\frac{f'_{df}}{f'_{dvh}}\right] \quad \text{for } q_f = q_v = q_{fv} \tag{3.43.8}$$

The three cases of design interest are summarized as follows:

$$0 < L \leq L_{fv} \qquad q_{max.} = \frac{b\ D\ f'_{dvh}}{108\ B\ L}$$

Shear controls

$$L = L_{fv}$$

$$q_f = q_v \qquad q_{max.} = \frac{b\ D\ f'_{dvh}}{108\ B\ L} = \frac{b\ D^2 f'_{df}}{1296\ B\ L^2} \tag{3.43.9}$$

$$L \geq L_{fv} \qquad q_{max.} = \frac{b\ D^2 f'_{df}}{1296\ B\ L^2}$$

Flexure controls

Prior to computing resistance functions from Equation 3.43.3 and 3.43.7, a further simplification can be introduced by recognizing that the factor which is used to convert conventional timber working stresses to dynamic yield stresses has a constant value of 4.0 (Chapter 2). Thus, by factoring this value, we obtain simplified expressions for the resistance function

$$L \leq L_{fv} \qquad \frac{q_v\ B\ L}{f_{vh}} = \frac{b\ D}{27} \tag{3.43.10}$$

$$L \geq L_{fv} \qquad \frac{q_f\ B\ L^2}{f_f} = \frac{b\ D^2}{324} \tag{3.43.11}$$

Computed values of the resistance functions $q_f\ B\ L^2/f_f$ and $q_v B\ L/f_{vh}$ are supplied in Table 3-65 for standard sizes of timber beams. However, since the ratio f_f/f_{vh} for common structural timbers varies over an appreciable range, it is necessary to evaluate L_{fv} for each proposed application. The design procedure involves computing the resistance function (either $q_v B\ L/f_{vh}$ or $q_f BL^2/f_f$, according to the designer's best judgment as to whether $L < L_{fv}$ or $L > L_{fv}$), then selecting a beam which will furnish the required resistance, and finally computing L_{fv} for the beam by use of Equation 3.43.8 to verify that the assumed failure mode was actually critical.

Table 3-65

RESISTANCE FUNCTIONS FOR SIMPLY-SUPPORTED AND FIXED-END TIMBER BEAMS

NOMINAL SIZE OF BEAM, in. b x D	SIMPLY-SUPPORTED BEAMS $L_{fv} = \frac{D}{12}\frac{f'_{df}}{f'_{dvh}} = \frac{D}{12}\frac{f'_f}{f_{vh}}$		FIXED-END BEAMS $L_{fv} = \frac{D}{8}\frac{f'_{df}}{f'_{dvh}} = \frac{D}{8}\frac{f_f}{f_{vh}}$	
	Shear Resistance Function($L < L_{fv}$) $[q_v BL/f_{vh}]$	Flexure Resistance Function($L > L_{fv}$) $[q_f BL^2/f_f]$	Shear Resistance Function($L < L_{fv}$) $[q_v BL/f_{vh}]$	Flexure Resistance Function($L > L_{fv}$) $[q_f BL^2/f_f]$
4 x 4	0.487	0.147	0.487	0.221
4 x 6	0.755	0.346	0.755	0.519
4 x 8	1.007	0.629	1.007	0.944
4 x 10	1.276	1.010	1.276	1.515
4 x 12	1.544	1.480	1.544	2.220
4 x 14	1.813	2.040	1.813	3.060
4 x 16	2.081	2.688	2.081	4.032
4 x 18	2.350	3.428	2.350	5.142
6 x 6	1.120	0.513	1.120	0.770
6 x 8	1.528	0.955	1.528	1.433
6 x 10	1.935	1.532	1.935	2.298
6 x 12	2.343	2.245	2.343	3.368
6 x 14	2.750	3.094	2.750	4.641
6 x 16	3.157	4.078	3.157	6.117
6 x 18	3.565	5.199	3.565	7.799
8 x 8	2.083	1.302	2.083	1.953
8 x 10	2.639	2.089	2.639	3.134
8 x 12	3.194	3.061	3.194	4.592
8 x 14	3.750	4.219	3.750	6.329
8 x 16	4.306	5.562	4.306	8.343
8 x 18	4.861	7.089	4.861	10.634
10 x 10	3.343	2.647	3.343	3.971
10 x 12	4.046	3.877	4.046	5.816
10 x 14	4.750	5.344	4.750	8.016
10 x 16	5.454	7.045	5.454	10.568
10 x 18	6.157	8.979	6.157	13.469
12 x 12	4.898	4.694	4.898	7.041
12 x 14	5.750	6.469	5.750	9.704
12 x 16	6.602	8.528	6.602	12.792
12 x 18	7.454	10.870	7.454	16.305
14 x 14	6.750	7.594	6.750	11.391
14 x 16	7.750	10.010	7.750	15.016
14 x 18	8.750	12.760	8.750	19.141
14 x 20	9.750	15.844	9.750	23.766

As used in Equation 3.43.10 and 3.43.11, f_{vh} and f_f refer to conventional working stresses in horizontal shear and in flexure, as proposed in standard stress-grading codes for structural timber.

(2) Fixed-End Beam

Analytical equations are similarly developed for the fixed-end timber beam. Any design based upon the assumption of end fixity must, however, recognize the practical difficulties which may result from a requirement for full restraint of the beam ends. The maximum elastic moment occurs at the fixed ends, and can be expressed as

$$M = \frac{q_f \times 12\ B \times 144\ L^2}{12} = 144\ q_f\ B\ L^2$$

Rearranging terms, and expressing the equation in terms of the loading q_f associated with dynamic yield stress in flexure (f'_{df}) for a rectangular timber beam, we obtain

$$\frac{q_f\ B\ L^2}{f'_{df}} = \frac{b\ D^2}{864} \qquad (3.43.12)$$

The maximum horizontal shear occurs at the fixed end, hence the expression for the loading q_v associated with dynamic yield stress in horizontal shear (f'_{dvh}) is the same as for the simply-supported case.

$$\frac{q_v\ B\ L}{f'_{dvh}} = \frac{b\ D}{108} \qquad (3.43.7)$$

Equations 3.43.12 and 3.43.7 can be solved simultaneously to find the length of loaded fixed-end beam (L_{fv}) at which yield stresses are simultaneously developed in flexure and in horizontal shear.

$$L_{fv} = \frac{D}{8}\left[\frac{f'_{df}}{f'_{dvh}}\right] \quad \text{for} \quad q_f = q_v = q_{fv} \qquad (3.43.13)$$

The three cases of design interest are summarized as follows:

$0 < L \leq L_{fv}$ $\qquad q_{max.} = \dfrac{b\ D\ f'_{dvh}}{108\ B\ L}$

Shear controls

$L = L_{fv}$ $\qquad q_{max.} = \dfrac{b\ D\ f'_{dvh}}{108\ B\ L} = \dfrac{b\ D^2 f'_{df}}{864\ B\ L^2}$

$q_f = q_v$ (3.43.14)

$L \geq L_{fv}$ $\qquad q_{max.} = \dfrac{b\ D^2 f'_{df}}{864\ B\ L^2}$

Flexure controls

As explained in the discussion of the simply-supported timber beam, however, these dynamic-loading resistance functions may also be expressed in terms of conventional timber working stresses. Factoring the conversion factor of 4.0, the resistance functions can be expressed in terms of conventional working stresses.

$$L \leq L_{fv} \qquad \frac{q_v\ B\ L}{f_{vh}} = \frac{b\ D}{27} \tag{3.43.15}$$

$$L \geq L_{fv} \qquad \frac{q_f\ B\ L^2}{f_f} = \frac{b\ D^2}{216} \tag{3.43.16}$$

The terms f_{vh} and f_f in Equations 3.43.15 and 3.43.16 again refer to conventional working stresses in horizontal shear and in flexure, as proposed in standard stress-grading codes for structural timber. Resistance functions computed from these equations can be used directly in design by first assuming the controlling function for a specific beam ($L \leq L_{fv}$ or $L \geq L_{fv}$), then selecting a suitable beam, and finally computing L_{fv} from Equation 3.43.13 to check the validity of the initial assumption as to controlling failure mode.

Table 3-65 contains values of shear and moment resistance functions, computed for standard sizes of timber beams, which are applicable both to simply-supported and to fixed-end conditions. The resistance functions for the flexural mode, as presented in this table, are computed by

assuming the larger dimension of a rectangular cross-section as its depth, D, and its smaller dimension as the beam width, b.

The cost of structural timber elements is a function of the number of board feet of timber contained in the element. In general, the smaller the dimension of timber used in the assembly of an element, the lower the material cost. However, it should be noted that often increased fabricating costs will offset this initial material cost advantage.

The total cost of any timber element may be expressed:

$$C_T = \frac{X_w b\ D\ L}{1000} \tag{3.43.17}$$

Where:

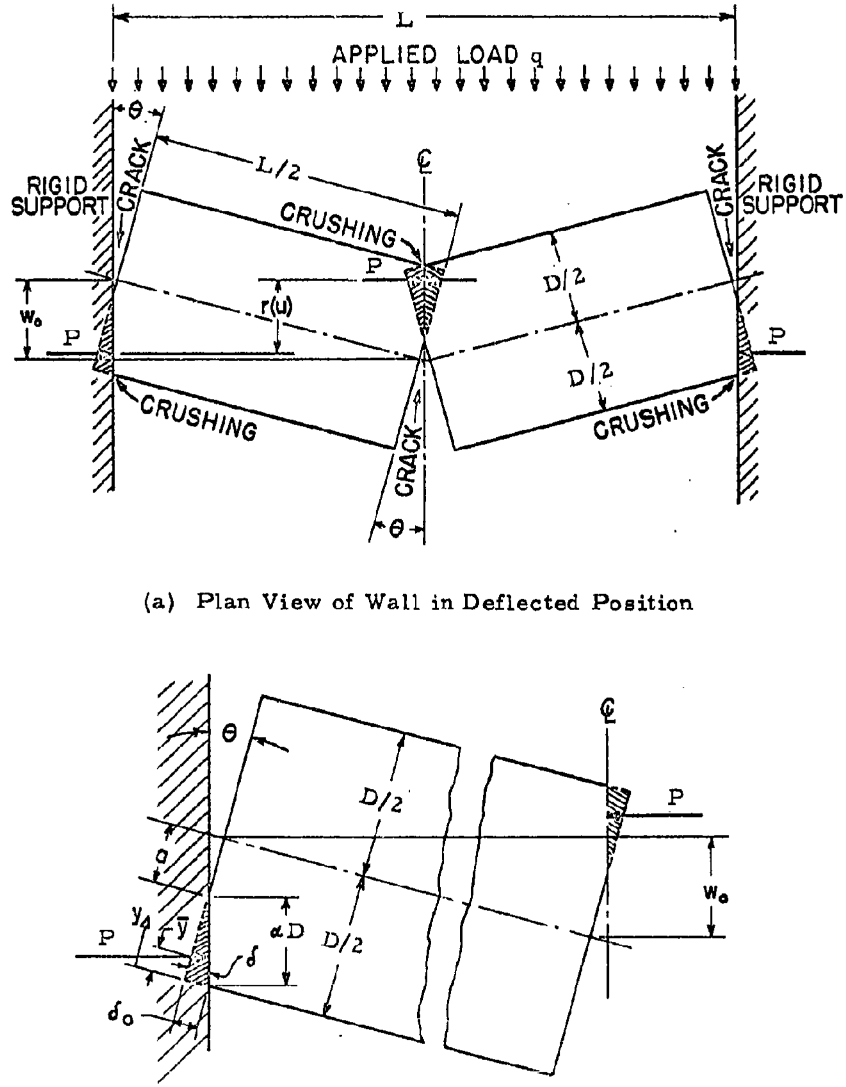
X_w = unit cost of timber (MBF)
b = width of members (in.)
D = depth of (in.)
L = length of member (ft.)

3.5 Masonry Walls

Under certain conditions, masonry walls can withstand much larger lateral loads than those predicted by conventional analyses based on simple bending. This additional strength is developed in walls of appreciable thickness whose ends are constrained by supports which are essentially unyielding. For example, in experiments on brick beams [47], it was found that beams with rigid supports developed from three to six times the load-carrying capacity of simply supported beams. A so-called "arching action" takes place, in which the resistance of the wall to lateral loads is due entirely to opposing forces which are set up in the plane of the panel. The magnitude of the total opposing force is directly related to the resistance of the masonry material to crushing at the midspan and at the supported ends.

The idealized wall, as analyzed in this section [15, 16], can consist of brick, concrete rubble masonry, unreinforced concrete, concrete block, or similar materials with appreciable thickness and compressive strength. The wall is analyzed as a simply-supported beam whose supports remain fixed in position during the loading of the beam. The beam is assumed to be of uniform solid cross-section, with a clear span of L feet, a depth of D inches, and a unit one-inch width. It is shown in a deformed position in Figure 3-6, with details of the geometry at the contact area. The following nomenclature is indicated on these figures.

D	=	thickness of wall, (in.)
L	=	clear span of wall, (ft)
w_o	=	maximum deflection of center line of wall for a given angular rotation θ, (in.)
θ	=	angle of rotation of half-wall
a	=	portion of half-wall thickness $D/2$, measured from the center line of the wall for a given horizontal rotation θ, which is no longer in contact with the support, (in.)
$\alpha \frac{D}{2}$	=	length of contact area corresponding to a given angular rotation θ
y	=	coordinate measured as indicated in Figure 3-6, (in.)



(a) Plan View of Wall in Deflected Position

(b) Geometry at Support

Figure 3-6

IDEALIZED MASONRY WALL

δ = shortening at distance y of material in contact with support (in.)

δ_o = maximum shortening at extreme fiber of material in contact with support, (in.)

The assumed mode of response of the idealized wall is such that each half of the beam rotates as a rigid body about the first point in contact with the support. Equilibrium requires that the contact areas at the ends and center of the wall be equal. The motion is such that the length of the contact area decreases with increasing center deflection. For small values of angular rotation θ the distance a can be expressed as

$$a = 3\,L\left[\frac{1 - \cos\theta}{\sin\theta}\right] \tag{3.5.1}$$

The maximum center line deflection can be expressed in terms of the rotation of the center line of the beam.

$$w_o = 2\,a\,(1 - \cos\theta) + \frac{12\,L}{2}\sin\theta$$

or (3.5.2)

$$w_o = 12\,L\left[\frac{1 - \cos\theta}{\sin\theta}\right]$$

The decrease in the length of contact area and the increase in maximum center deflection are thus related, for the specific units, as

$$w_o = 4\,a \tag{3.5.3}$$

It is convenient to introduce the following nondimensional notation.

$$\left.\begin{aligned} u &= \frac{w_o}{D} \\ \text{and} \qquad & \\ n &= \frac{D}{12\,L} \end{aligned}\right\} \tag{3.5.4}$$

then

$$\left.\begin{aligned} \sin\theta &= \frac{2\,nu}{1+(nu)^2} \\ \cos\theta &= \frac{1-(nu)^2}{1+(nu)^2} \end{aligned}\right\} \tag{3.5.5}$$

From the geometry of Figure 3-6, the fraction α of the half-depth $D/2$ which is in contact with the support for an angular rotation θ is given by

$$\alpha = \frac{1}{2}\left[\frac{1+(nu)^2}{1-(nu)^2}\right]\left[1-\frac{u}{2}\right] \tag{3.5.6}$$

The total shortening of the material at a position y is

$$\delta = \frac{nu\,D\left(1-\frac{2y}{D}-\frac{u}{2}\right)}{1-(nu)^2} \tag{3.5.7}$$

Since the beam is assumed to be cracked at its center (see Figure 3-6) the total change in length δ given by Equation 3.5.7 is averaged over one-half of the beam length to find the average strain. Thus,

$$e_{avg.} = \delta/6\,L \tag{3.5.8}$$

It will be assumed that the strain along any one fiber of the beam varies linearly from a maximum at its contact end to zero at its cracked end. The strain at the contact end is then

$$e_{cm} = 2\,e_{avg} = \frac{\delta}{3\,L} \tag{3.5.9}$$

The strain at the contact area in a beam fiber a distance y from the unloaded wall surface can then be expressed as,

$$e_{cm} = 4n^2u\;\frac{\left(1-\frac{2y}{D}-\frac{u}{2}\right)}{1-(nu)^2} \tag{3.5.10}$$

Defining f'_{cm} as the ultimate crushing strength associated with the ultimate strain e'_{cm} for the material, and introducing the nondimensional variable R where

$$R = \frac{e'_{cm}}{4n^2} = \frac{36\, e'_{cm}\, L^2}{D^2} , \tag{3.5.11}$$

Eq. 3.5.10 can be written as,

$$\frac{R\, e_{cm}}{e'_{cm}} = u \frac{\left[1 - \frac{2y}{D} - \frac{u}{2}\right]}{1 - (nu)^2} \tag{3.5.12}$$

For the range of interest considered here, both n and u will be sufficiently small so that the term $\left[1 - (nu)^2\right]$ can be approximated by unity. The equation for strain variation from the ultimate along a unit width of contact area is then given by,

$$\frac{R\, e_{cm}}{e'_{cm}} = u \left[1 - \frac{2y}{D} - \frac{u}{2}\right] \tag{3.5.13}$$

The arching force per unit width of beam, P, represents the resultant of stress distribution along the contact areas at the end and center of the beam, and is related to the nondimensional center deflection, u. The force P has been evaluated for several values of R. The results are plotted in Figure 3-7 in terms of the dimensionless parameters u and $8\, P/(f'_{cm} D)$.

The moment resistance, M, is defined as the moment due to arching forces, $M = r(u)\, P$, where $r(u)$ is the moment arm shown in Figure 3-6. From the geometry of the deflected wall,

$$r(u) = D \left[\frac{1 + (nu)^2}{1 - (nu)^2}\right] \left[1 - \frac{2\bar{y}}{D} - \frac{u}{1 + (nu)^2}\right] \tag{3.5.14}$$

where $\bar{y}$ locates the centroid of the stress distribution along the contact area. If, as before, the quantity $(nu)^2$ is neglected in comparison with unity, an approximate form for $r(u)$ is found to be

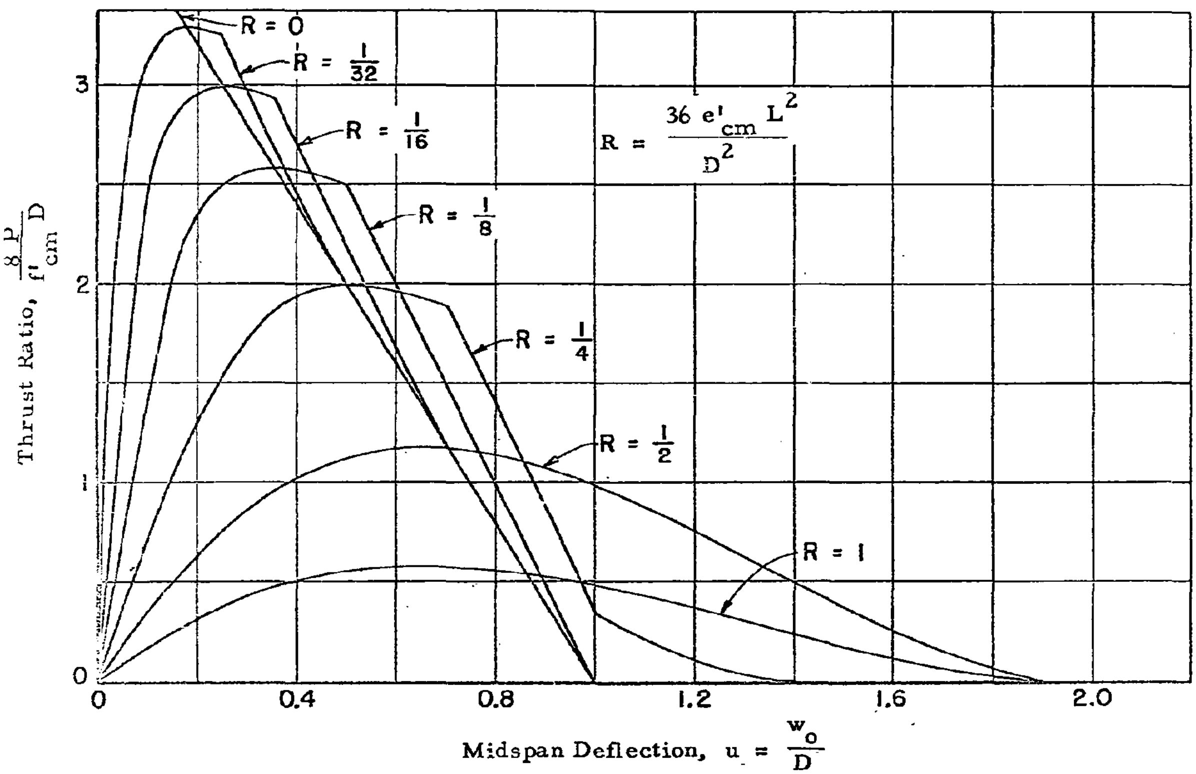


Figure 3-7

VARIATION OF THRUST RATIO WITH MID-SPAN DEFLECTION,
IDEALIZED MASONRY WALL

$$r(u) = D \left(1 - u - \frac{2\bar{y}}{D}\right) \qquad (3.5.15)$$

The moment resistance, M, has been computed for several values of R and the results are plotted in Figure 3-8 in terms of the dimensionless parameters u and $16M\,(f'_{cm}\,D^2)$.

Table 3-66 can be used to obtain solutions for the ultimate transverse unit load q for specified values of wall length L, wall thickness D, and masonry crushing strength f'_{cm}. Conversely, if q is specified, the required wall thickness D can be obtained. Assuming that D and L are known, the quotient 12 L/D is first calculated. Next, for a specified value of f'_{cm}, the ultimate strain e'_{cm} is calculated as the quotient f'_{cm}/E. The resistance function $R = e'_{cm}/4 \times (12\ L/D)^2$ can now be calculated. Entering the table with values for R and for 12 L/D, the corresponding value for q/f'_{cm} can be read directly. The table includes the assumption that the modulus of elasticity, E, is known for the wall material.

Figure 3-9 shows ultimate values of lateral load (psi), plotted as ordinate with half-span length, $\frac{L}{2}$ (ft) as abscissa, for selected ratios of $w_o/12\ L$. Figure 3-10 shows plotted values of the total lateral wall thrust, P (kips/sq in.), which is developed at at the rigid wall supports for selected ratios of $w_o/12\ L$.

The cost of concrete masonry units includes the cost of the block and of No. 3 steel reinforcing rod which is grouted in place.

$$C_t = X_{cm}$$

$$C_T = [HL]\ X_{cm}$$

Where: X_{cm} = unit cost of reinforced concrete masonry units, (\$/sq in.)

L = length of wall, (ft)

C_t = cost factor for block wall, \$/sq ft)

C_T = total cost of block wall, (\$)

H = height of wall (ft)

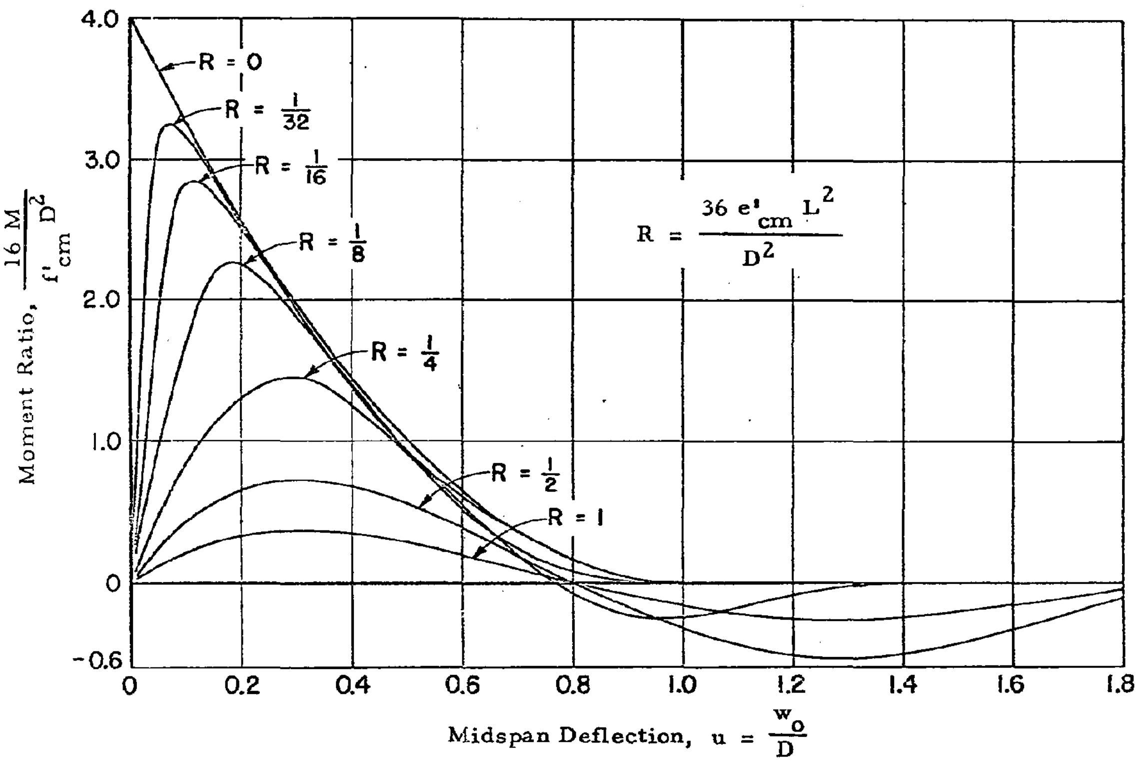


Figure 3-8

VARIATION OF RESISTING MOMENT RATIO WITH MID-SPAN DEFLECTION, IDEALIZED MASONRY WALL

Table 3-66

RATIO OF ULTIMATE UNIT TRANSVERSE LOAD TO ULTIMATE UNIT CRUSHING STRENGTH FOR ARCHING MASONRY WALL (q/f'_{cm})

Value of Non-Dimensional R						Ratio
1.000	0.5000	0.2500	0.1250	0.0625	0.03125	12 L/D
0.1808	0.3616	0.7220	1.1320	1.4255	1.626	1.0
0.08033	0.1607	0.3209	0.5031	0.6336	0.7227	1.5
0.04519	0.09039	0.1805	0.2830	0.3564	0.4065	2.0
0.02892	0.05785	0.1155	0.1811	0.2281	0.2602	2.5
0.02008	0.04017	0.08022	0.1258	0.1584	0.1807	3.0
0.01476	0.02951	0.05894	0.09241	0.1164	0.1327	3.5
0.01130	0.02260	0.04513	0.07075	0.08909	0.1016	4.0
0.008926	0.01785	0.03565	0.05590	0.07040	0.08030	4.5
0.007230	0.01446	0.02888	0.04528	0.05702	0.06504	5.0
0.005975	0.01195	0.02387	0.03742	0.04712	0.05375	5.5
0.005021	0.01004	0.02006	0.03144	0.03960	0.04517	6.0
0.004278	0.008557	0.01709	0.02679	0.03374	0.03849	6.5
0.003689	0.007379	0.01473	0.02310	0.02909	0.03318	7.0
0.003213	0.006428	0.01284	0.02012	0.02534	0.02891	7.5
0.002824	0.005649	0.01128	0.01769	0.02227	0.02541	8.0
0.002502	0.005004	0.009993	0.01567	0.01973	0.02251	8.5
0.002231	0.004464	0.008914	0.01398	0.01760	0.02007	9.0
0.002003	0.004006	0.008000	0.01254	0.01580	0.01802	9.5
0.001808	0.003616	0.007220	0.01132	0.01426	0.01626	10.0
0.001639	0.003279	0.006549	0.01027	0.01293	0.01475	10.5
0.001494	0.002988	0.005967	0.009355	0.01178	0.01344	11.0
0.001367	0.002734	0.005459	0.008560	0.01078	0.01229	11.5
0.001255	0.002511	0.005014	0.007861	0.009899	0.01129	12.0
0.001070	0.002139	0.004272	0.006698	0.008435	0.009621	13.0
0.0009222	0.001845	0.003684	0.005776	0.007273	0.008296	14.0
0.0008033	0.001607	0.003209	0.005031	0.006336	0.007227	15.0
0.0007061	0.001412	0.002820	0.004422	0.005568	0.006352	16.0
0.0006254	0.001251	0.002498	0.003917	0.004933	0.005626	17.0
0.0005579	0.001116	0.002228	0.003494	0.004400	0.005019	18.0
0.0005007	0.001002	0.002000	0.003136	0.003949	0.004504	19.0
0.0004519	0.0009039	0.001805	0.002830	0.003564	0.004065	20.0
0.0004099	0.0008198	0.001637	0.002567	0.003232	0.003687	21.0
0.0003735	0.0007470	0.001492	0.002339	0.002945	0.003360	22.0
0.0003417	0.0006835	0.001365	0.002140	0.002695	0.003074	23.0
0.0003138	0.0006277	0.001253	0.001965	0.002475	0.002823	24.0

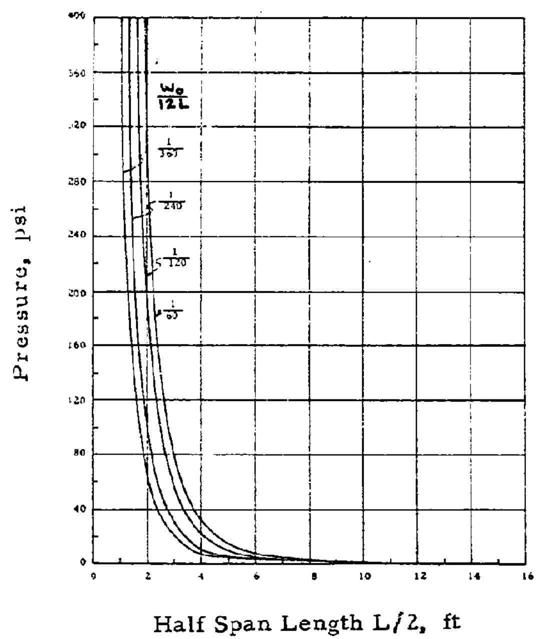


Figure 3-9

ULTIMATE UNIT TRANSVERSE LOAD ON WALL

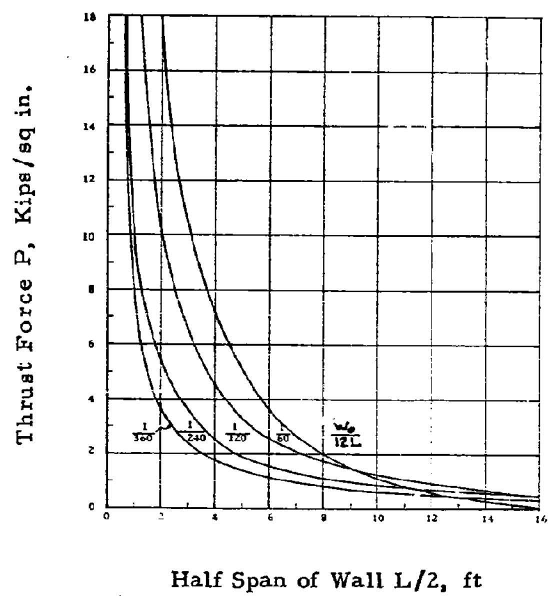


Figure 3-10

WALL THRUST AT RIGID SUPPORTS

3.6 Miscellaneous

3.61 Introduction

The earlier sections of this chapter have given detailed attention to the design of steel, reinforced-concrete and timber structural elements for possible use in buried, blast-resistant shelters. The importance of these materials in the construction industry has warranted their thorough examinations. This final section describes, in greatly abbreviated form, a few of the many specialized materials and combinations with possible application to shelter design: no detailed analyses or supporting calculations are provided for these cases.

3.62 Prestressed Concrete

The prestressing of concrete beams appears to offer no major advantages in the design of heavy, blast-resistant structures. Prior to any ultimate moment failure, tension cracks will open in the prestressed concrete. Once this condition has been reached, the prestressed beam behaves essentially the same as a normally-reinforced beam. Failure will occur at the same load level, without regard to prestressing. Prestressing finds a favorable range of application when the live loading is eight times the dead loading, or less. Live loadings of 10 to 100 times the dead load, which is the loading range under consideration, are unsuitable for the use of prestressed concrete.

3.63 Precast Concrete

A relatively-large proportion of the in-place cost of reinforced concrete is attributable to the cost of formwork. This is indicated in Table 2-9 and becomes apparent in the design examples of Chapter 4. For this reason, any means which promotes the interchange or re-use of forms will reduce the total cost of concrete structures. Where the concrete elements are not excessively large, and where hauling distance from the casting bed to the construction sites is sufficiently short, the selective use of precasting techniques might very well reduce structural costs for reinforced concrete shelters. Also, apart from reusability of forms, a properly-organized precasting operation should result in better concrete control and in a more efficient use of labor.

3.64 Reinforced Concrete Joist Systems

Concrete joist systems, either one-way as in "T-beam" designs or two-way as in waffle slabs, are successfully employed in conventional construction. A joist system is frequently used for floors and roofs, since it can furnish moment resistances equivalent to that supplied by a solid slab with an appreciable reduction in dead load. Offsetting this saving, in part at least, is an increase in forming costs and a reduction in the shearing mode resistance as compared with a solid slab.

In conventional construction, where moment normally governs design and where dead load is an important factor, the advantages of a joist system frequently outweigh its disadvantages. This is particularly true when joists are precast, or where reusable forms are introduced. In blast resistant design, where heavy loads and short spans predominate, a minor reduction in dead load is of little consequence, and the shearing resistance of the system is frequently critical. As an additional factor, the solid slab may provide more protection from radiation than is supplied by a joist system of equivalent strength.

3.65 Composite Construction

Composite construction is used where long spans, with large positive moments and deflections, govern design. However, for heavy blast loading, shear is frequently the governing design parameter. The shearing strength of the concrete in composite steel-concrete design is small, particularly since the concrete will probably crack before the full shear resistance is developed in the web of the steel structural shape. For such cases, the concrete would not contribute to the load carrying capacity of the steel member. Composite design would also increase the practical difficulties in obtaining full end-fixity for the member, with its resulting economies. In summary, it is concluded that composite beam design offers no advantages for heavy blast resistant structures.

3.66 Stabilized Earth

The ability of selected earth materials to resist applied loading can be increased by properly-selected stabilization methods. These can include

compaction, mixture with other soils, chemical additives, drainage, electrolysis, addition of binder materials, etc. The unsolved problem, as repeatedly discussed in this report, is the development of a relationship between soil strength, depth of burial for a structure, magnitude of overpressure at the ground surface, and required structural resistance. This interrelationship is considered in more detail in Appendix A to this report.

Soil stabilization can increase soil shearing resistance. At the same time, the stabilization process can be expected to alter the elastic characteristics of the soil and thus modify its response to an induced groundshock. The increase in the combined strength of a particular soil and a particular structure may, if an economic study could be made, fully justify the cost of stabilization. Indeed, for favorable situations of blast wave and burial depth, the soil itself might conceivably constitute the shelter structure (see Appendix A). Lacking any real understanding of the quantitative significance of soil strength in the soil-structure-blast wave interaction, however, it seems pointless to discuss the relative costs and effectiveness of soil stabilization processes.

3.67 Fiber Reinforced Plastics

Increasing interest is currently shown in the use of plastics for structural purposes. Of the many varieties of available plastics, the most suitable structural type appears to be fiberglass reinforced plastic (FRP). This material is reinforced with glass fiber in the form of cloth, woven roving fibers, or chopped fibers. Various methods are used to combine the fiber with a matrix of epoxy or polyester resin, and to form the desired structural shape.

It is concluded that the spray-up polyester laminate, using chopped glass fibers with a polyester resin and forming structural shapes by spray applications, is of primary interest for shelter construction. Surfaces of complex curvatures can thus be formed at minimum forming costs. However, this method of application results in thickness variations for the finished product.

The FRP structure, in its present state of development, does not constitute the least-cost structural form for a buried shelter. However, it has certain intrinsic advantages (see Appendix B) which may render its use more feasible. The material warrants future re-examination, particularly as additional experience in its fabrication and performance is accumulated.

CHAPTER 4 STRUCTURAL DESIGNS FOR THE 100-MAN SHELTER

4.1 Introduction

Chapter 3 supplies extensive data pertaining to the design and costing of structural elements. In order to evaluate structural costs for entire shelters, however, these elements must be utilized into actual shelter designs. Obviously, there are a great many ways in which structural elements of different materials can be combined to form a complete structure. The results of other investigations[9, 10] supply some guidance as to optimum configuration and interior layout for buried shelters at selected levels of overpressure. Other studies[2, 9, 48] also indicate a relationship between optimum configuration and level of overpressure.

From the findings of these earlier studies, and from the cost and analytical relationships developed in this report, representative shelter configurations and material combinations are selected for detailed study. The paramount consideration in making these selections is minimum structural cost for the total shelter. However, acceptable shelter designs must also utilize construction methods and materials which lend themselves to a mass shelter construction program. The shelters are designed for 100-man capacity, and their structural requirements are investigated over a wide range of overpressures. Similar design and costing procedures could readily be applied to other sizes of shelter, if subsequently desired. Shelter costs presented in this chapter are based on theoretical design material quantities. In many cases limitations on size or material availability dictate that substitutions or slight revisions of the material quantities be made. These changes normally tend to increase cost to a minor degree.

4.2 Design Assumption

4.21 <u>Area and Volume Requirements</u>

All shelter configurations are based on 100-man capacities. Shelter floor area is determined by bunk layout sufficient to sleep 100 people simultaneously, plus a minimum of 250 square feet administrative and service area.

The bunking recommendations listed in Reference 9 are used, except that bunk width has been increased from 2'-2" to 2'-6". Alternatively, if desired, the 2'-2" bunk width could be retained and the aisle width increased above the 2'-0" specified minimum. The bunking criteria used in this study are summarized as follows:

Bunk dimensions	- 6' 4" long x 2' 6" wide
Maximum tiering	- 5 high
Lower bunk clearance	- 7"
Vertical bunk spacing	- 1' 8"
Clearance between bunk and ceiling	- 1' 8"
Minimum aisle width	- 2' 0"

Minimum acceptable floor area and shelter volume are 800 sq ft and 6500 cu ft, respectively, corresponding to 8 sq ft/man and 65 cu ft/man.

4.22 Dimensional Limitations

The shelters are designed as a "survival" mechanism, where structural costs are held to a minimum. The limiting structural dimensions, as specified herein, reflect this design concept.

a) Minimum concrete shell thickness = 3"
b) Minimum structural concrete slab thickness = 4"
c) Minimum non-structural concrete slab thickness = 2"
d) Minimum poured-in-place concrete wall thickness = 6"
e) Minimum concrete footing thickness = 8"
f) Recommended concrete covers, measured from the centroid of steel reinforcement, are listed in Table 4-1

Table 4-1

RECOMMENDED MINIMUM CONCRETE COVER FOR REINFORCED CONCRETE MEMBERS

Total Depth of Member	Type of Member: Beams, Slabs and Shells	Type of Member: Walls and Column
$D \leq 6$	3/4"	3/4"
$6 > D \leq 8$	1"	
$8 > D \leq 10$	1-1/2"	
$D > 10$	2-1/2"	
$6 < D \leq 12$		1"
$D > 12$		0.1D

f) Ratio of compression to tension steel, θ', has a minimum value of 0.25.

g) Tensile steel percentage, ϕ_c = 0.25 minimum, in concrete flexural members where

$$\phi_c = \frac{100\,A'_s}{bd}$$

h) Main steel percentage, ϕ_t = 0.50 minimum, in axially-loaded concrete compression members where

$$\phi_t = \frac{100\,(A'_s + A''_s)}{bD}$$

i) Main steel percentage, ϕ'_t = 0.50 minimum, in concrete compression members resisting bending moments due to eccentric axial loads or lateral loads where

$$\phi'_t = 100\left[\frac{A'_s + A''_s}{bd}\right]$$

j) Percentage of temperature steel, ϕ_{te} = 0.10 minimum, where

$$\phi_{te} = 100\left[\frac{A_s\,(\text{temp.})}{bD}\right]$$

in any concrete member where temperature steel is specified.

k) Minimum footing width = 2'-0".

l) Minimum depth of burial for roof of shelter = 3'-6".

4.23 <u>Burial Requirements</u>

Depth of burial must satisfy both full burial and radiation attenuation criteria. The criteria for full burial[1], which are illustrated in Figure 1-1,

require only that the roof of a cubicle be located at or below ground level. Curved roof surfaces must have a minimum earth cover satisfying the larger of the following:

a) Cover of $S_L/8$ over the crown.

b) Average cover of $S_L/4$.

Since this study is limited to rectangular, full circle or semi-circular structures, the controlling full-burial requirements can be expressed as follows:

Rectangular structures:	$h_{min.}$ = zero
Arch and cylinder:	$h_{min.} = 0.143 S_L$
Dome and sphere:	$h_{min.} = 0.125 S_L$

where

S_L = diameter of curved structure, (ft)

$h_{min.}$ = minimum depth of cover over the highest point of the structure, (ft)

The depth of burial which is required for radiation protection is related to the types and initial intensity of external radioactive fields, to their decay as a function of time, and to the tolerable radiation dosage for a shelter occupant. Since this study is primarily concerned with structural materials, several broad assumptions are applied in obtaining estimates of burial requirements for shelters. The external fields are classified as "initial" and "residual," with differing mean energy characteristics, and are separately examined for their effect on a structure.

The initial radiation is considered to include alpha particles, beta particles, gamma emanations and neutrons. The alpha and beta particles, because of their limited range and effectiveness, do not affect structural material requirements for buried shelters. Neutrons are considered as directly additive to gamma radiations, insofar as this analysis is concerned, despite some differences in their shielding requirements. Residual radiation

will consist primarily of lower energy-level gamma radiations, postulated to have a maximum one-hour intensity of 10,000 roentgens per hour (R/hr).

The tolerable dosage for humans is still a subject of study, although various recommendations have been offered.[3, 4, 6, 49] The body area which is exposed to radiation, as well as the time period over which such radiation is received, will influence the tolerable dose. Without proposing absolute limits for safe dosage, it appears unwise to design shelter shielding so that the initial dose plus the residual accumulated dose during a two-week shelter stay will equal the maximum safe effective dose. It must be recognized that, in the majority of practical cases, a person would receive an additional dose after leaving the shelter.

In this study, shielding requirements are primarily related to the level of initial radiation and to a permissible effective dose of 50 R. For many cases, particularly where the level of initial radiation is high, the additional effective dose within the shelter due to residual radiation is negligible. However, this effect is checked by assuming a one-hour reference dose rate of 10,000 R/hr and a two-week shelter stay. The shelter is analyzed as a plane shield exposed to radiation, and required shielding is expressed as equivalent feet of earth cover. A minimum cover of 3'-6" of earth over the structure is arbitrarily specified for all designs.

Table 4-2, prepared from data in References 3, 4, 6 and 49, lists design radiation levels for various weapon yields at selected overpressure levels. These data are plotted on Figure 4-1, with equivalent depth of cover as an ordinate.. Design equations A and B, as shown in Figure 4-1, are empirical approximations to the plotted data.

Equation A is of the form,

$$d_e = 3.5 + 0.029(p_{so} - 10) \qquad (4.23.1a)$$

Table 4-2

INITIAL RADIATION, RESIDUAL RADIATION AND EQUIVALENT DEPTH OF EARTH COVER FOR 50 R MAXIMUM EFFECTIVE DOSE AT SELECTED LEVELS OF OVERPRESSURE AND WEAPON YIELD

Peak Over-pressure Psi	W = 1 MT			W = 5 MT			W = 20 MT			W = 100 MT		
	Initial Radiation at Distance from G.Z.	Cumulative 2-Week Residual Radiation R	Depth of Earth Shielding ft	Initial Radiation at Distance from G.Z.	Cumulative 2-Week Residual Radiation R	Depth of Earth Shielding ft	Initial Radiation at Distance from G.Z.	Cumulative 2-Week Residual Radiation R	Depth of Earth Shielding ft	Initial Radiation at Distance from G.Z.	Cumulative 2-Week Residual Radiation R	Depth of Earth Shielding ft
200	9,500,000 R @ 2,500'	35,000	10.0*	1,200,000 R @ 4,300'	85,000	9.0*	375,000 R @ 6,800'	85,000	8.0	160,000 R @ 11,600'	85,000	7.5
130	4,000,000 R @ 2,900'	85,000	9.5*	370,000 R @ 5,000'	85,000	8.0*	170,009 R @ 7,900'	85,000	7.0	17,000 R @ 13,500'	85,000	5.0
100	1,800,000 R @ 3,400'	85,000	9.0*	105,000 R @ 5,800'	85,000	7.0	21,500 R @ 9,200'	85,000	5.5	1,500 R @ 15,800'	85,000	3.5
75	485,000 R @ 3,900'	85,000	8.3*	25,000 R @ 6,700'	85,000	5.5	5,200 R @ 10,500'	85,000	4.5	20 R @ 18,000'	85,000	3.5≠
50	162,000 R @ 4,400'	85,000	7.0*	8,000 R @ 7,500'	85,000	4.5	1,000 R @ 11,900'	85,000	3.5≠	------ @ 20,500'	85,000	3.5≠
25	7,800 R @ 6,300'	85,000	4.5	200 R @ 10,300'	85,000	3.5≠	------ @ 17,000'	85,000	3.5≠	------ @ 29,500'	85,000	3.5≠
10	75 R @ 10,000'	85,000	3.5≠	------ @ 17,000'	85,000	3.5≠	------ @ 27,000'	85,000	3.5≠	------ @ 46,500'	85,000	3.5≠

* Major portion (> 50%) of required thickness of earth shielding due to neutron emissions.

≠ Minimum of 3.5 cover, based primarily on residual radiation.

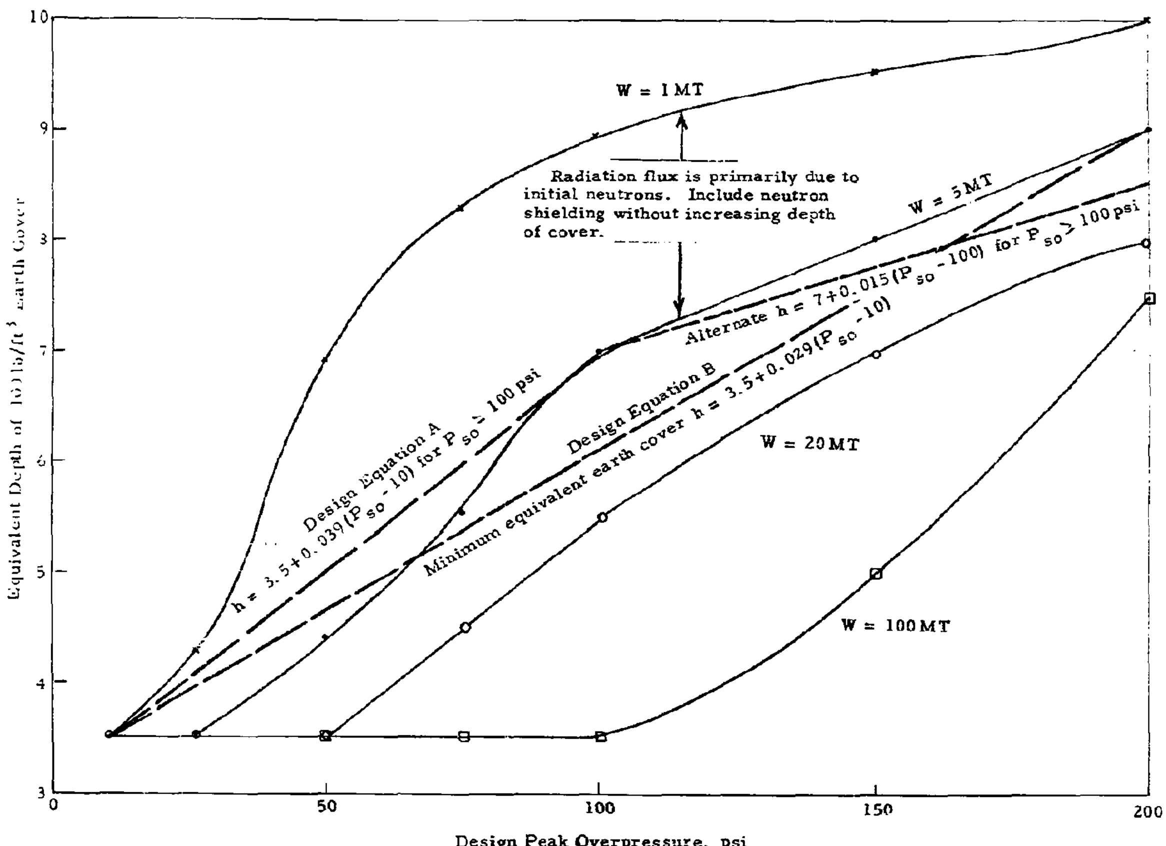


Figure 4-1

RELATION BETWEEN DESIGN PEAK OVERPRESSURE AND BURIAL REQUIRED FOR RADIATION PROTECTION
(Computed for Selected Weapon Yields, with Shelter Dose Limited to 50 R in Two-Week Period)

where

d_e = depth of equivalent earth cover, (ft)

p_{so} = design level of overpressure, (psi)

The static equivalent of Equation A can be expressed very approximately[1, 3] as

$$d_e = 3.5 + 0.020\, p_{static} \qquad (4.23.1b)$$

Equation B is of the form,

$$\left.\begin{array}{ll} (p_{so} \leq 100) & d_e = 3.5 + 0.039(p_{so} - 10) \\ (p_{so} \geq 100) & d_e = 7.0 + 0.015(p_{so} - 100) \end{array}\right\} \qquad (4.23.2a)$$

The static equivalent approximation of Equation 4.23.2a can be expressed as

$$\left.\begin{array}{ll} (p_{static} \leq 100) & d_e = 3.5 + 0.025\, p_{static} \\ (p_{static} \geq 100) & d_e = 6.0 + 0.01(p_{static} - 100) \end{array}\right\} \qquad (4.23.2b)$$

The former equation, primarily because of its simple form, is applied in the specific design examples which are studied in this chapter.

4.24 Excavation Requirements

The shape of the excavation for cubicle, arch, cylinder and dome shelter configurations is assumed to be the frustum of a pyramid or cone with 1:1 side slopes having the smaller base, A_1, equal to the exterior base area of the shelter. The smaller base area of the sphere configuration is assumed to be the required working area at the bottom of the excavation.

Using the end area method for determining volume,

$$\text{Volume} = \frac{z}{2}(A_1 + A_2) \qquad (4.24.1)$$

where

A_1 = gross plan area of the shelter (net floor area plus cross-sectional area of the walls) or the required working area at the base of a sphere or cylinder.

A_2 = area of excavation at ground level

z = depth of excavation

No allowance for shrinkage or swell is taken in determing back fill and haul requirements.

4.3 Cubicle

4.31 Introduction

The cubicle is extremely versatile in configuration and, since it employs conventional construction techniques, is perhaps the best understood of the configurations discussed in this study. A number of structural schemes are possible in the lower pressure ranges, and these are discussed in detail in Section 4.33 and 4.34. The paramount advantage of the cubicle, as compared with the shell configuration, is its relatively shallow burial requirement.

4.32 Layout Studies

The layout dimensions used in this study are based on a recent study of optimum bunking arrangements.[9] The shelter layouts utilize two primary bunking schemes, A and B, and two combinations, BA and BAA. These are shown in Table 4-3, where pertinent data are supplied regarding the four layout designs. The introduction of interior partitions into the shelter makes possible the further coupling of the layouts shown in this table.

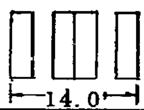
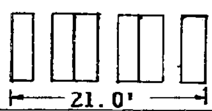
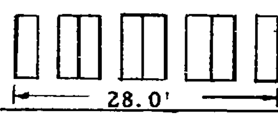
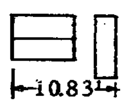
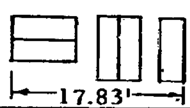
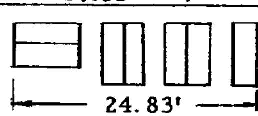
4.33 Design Alternatives

(a) Monolithic

One and two-way slabs are poured monolithically with their supporting walls. The advantages of this method are found in the simplicity of form work and the elimination of the "cumulative" effect inherent in all framing systems, whereby an applied load is successively transferred from

Table 4-3

BASIC LAYOUT DATA FOR 100-MAN CAPACITY CUBICLES

Style	Schematic Representation	Interior Bay Width(ft)	Interior Length(ft)	Interior Height(ft)	Net Area(sq ft)	Net Volume(ft^3)
A-Single		7.00	118.0	8.0	826	6600
A-Double		14.00	59.0	8.0	826	6600
A-Triple		21.00	40.0	8.0	840	6700
A-Quad		28.00	30.0	8.0	840	6700
B		10.83	82.0	7.5	890	6700
BA		17.83	49.0	7.5	875	6560
BAA		24.83	35.0	7.5	870	6500

one class of member to another. The disadvantages stem from the difficulty in providing easy access to all parts of the shelter. It is apparent that the optimum design of slabs is of prime importance in the economical design of monolithic structures.

(b) Flat Slab

This alternative is simply a modification of the monolithic design and makes possible the elimination of interior bearing walls without the introduction of overhead beams.

(c) Reinforced Concrete Framing Systems

Concrete beam and column systems can be employed, at least in the lower overpressure ranges. The advantage is that relatively inexpensive reinforced concrete masonry units can be used as walls between columns, thus eliminating expensive concrete bearing walls. The disadvantage arises from the relatively massive columns and beams which become necessary, since the beam and column must support the entire load on the effective span of the slab between supporting frames.

(d) Structural Steel Framing Systems

The advantages and disadvantages of reinforced concrete frames apply with equal force to steel framing systems.

(e) Timber Frame

Structural timber can be used in cubicle design but, because of low flexural limitations, its use would be limited to short spans and low overpressures.

(f) Foundations

Foundations are frequently of the raft variety, where a ground slab is designed to provide the same structural resistance as the roof slab. Alternatively, for favorable combinations of load and soil strength, a continuous or column footing may be found less costly. In either case, the choice is dictated by length of span and magnitude of overpressure. For the typical

soil considered in this study ($\phi = 15^{\circ}$, c = 2000 lb/sq ft), footing foundations are economical for clear spans greater than 12 feet and for overpressures less than 75 psi. For other cases, still considering this same soil, the ground slab or raft foundation is found to be preferable.

4.34 Sample Analysis and Cost Evaluation

In this section, sample designs from the five design alternatives listed in Section 4.33 are presented. These illustrate the design and cost procedures used in the study. It is suggested that these sample designs might also form the basis for analysis and cost development for actual shelter designs. The particular design and material parameters used in the trial design are based on the detailed analysis and costs criteria developed in Chapter 3 for specific structural elements.

TRIAL DESIGN 4.34 A

CONFIGURATION:

One story cubicle (see Figure 4-2)

STRUCTURAL SYSTEM:

Monolithic one-way slab - style A - single with one interior partition. (see Table 4-3)

DESIGN PARAMETERS:

q = 10 psi equivalent pressure including weight of slab and of earth cover

L = 7.0 ft clear span

(a) Roof Slab Design

From Table 3-15, for L = 7.0 ft and q = 10 psi, the minimum-cost structural parameters for the overhead one-way slab with fixed-end support are,

f_{dy} = 75,000 psi

f'_c = 3,600 psi

ϕ_c = 0.40 percent

ϕ_v = 0

D = 4.5 in.

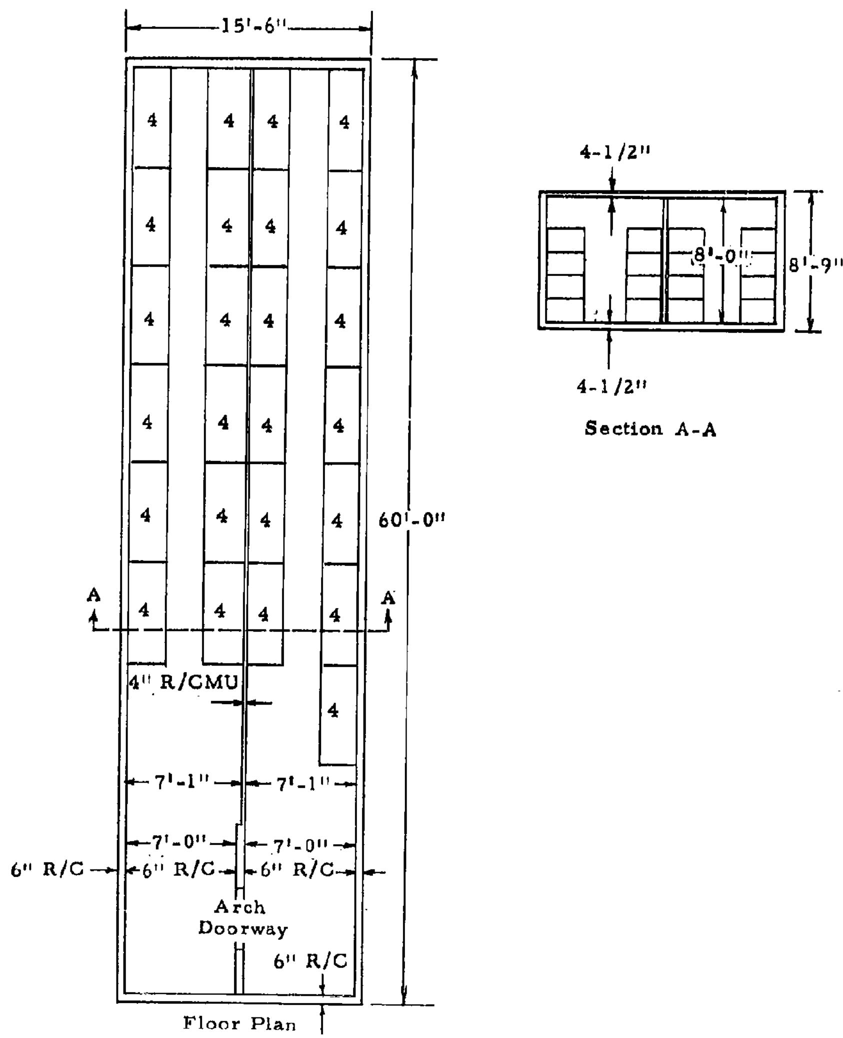


Figure 4-2

SHELTER LAYOUT TRIAL DESIGN 4.34A

It will be recalled that Table 3-15 includes the assumption that $d = 0.9\,D$. This approximation is reasonably correct for slabs of intermediate depth, but is somewhat in error for very thin or very deep slabs. Table 4-1 lists recommended minimum concrete covers for reinforced concrete members. While slab costs from Table 3-15 are used in the following design examples, more exact cost studies would involve a minor cost adjustment for cases where the $d = 0.9\,D$ assumption is not acceptable.

Check roof slab for "pure" shear, recalling that Table 3-15 incorporates the assumption that $d = 0.9\,D$.

Ultimate Shear Capacity

$$V_u = 0.22\ bd\ f'_c \qquad (3.33.6)$$

$$V_u = 0.22 \times 1.00 \times 0.9 \times 4.5 \times 3600 = 3210 \text{ lb/in. width.}$$

Maximum Shear on Slab

$$V = 6\ qL \text{ lb/in.width} \qquad (3.35.16b)$$

$$V = 6 \times 10 \times 7.0 = 420 < 3210$$

Only in rare instances will "pure" shear be a controlling factor, because of the high concrete strengths associated with economical slab design.

(b) Roof Slab Cost Factor

From Table 3-15

$$C_t = 1.66 \quad \$/\text{sq ft}$$

(c) Eccentrically-Loaded Side Wall Design

The bearing wall receives thrust and moment from the one-way slab. (See Section 3.35 for analysis.)

For use in Tables 3-41 to 3-44, inclusive, assume $f'_{dc} = 2500$ psi, based on recommendations of Section 3.35

$$\frac{q}{f'_{dc}} = \frac{10}{2500} = 0.004$$

$$q_d = \frac{1}{2}\left(\frac{\phi'_t}{100}\right)\left(\frac{f_{dy}}{f'_{dc}}\right)$$

The minimum wall dimension, $D = 6$ in., and the minimum percentage of reinforcement, $\phi_t = 0.50$, will almost certainly control design because of the short length of span (see Sections 3.35 and 4.22). For $\frac{q}{f'_{dc}} = 0.004$ and $\frac{d_{wall}}{L_{slab}} = \frac{0.9 \times 6.0}{7} = 0.77$, it is apparent from Tables 3-41 to 3-44, inclusive, that q_d (wall) $= 0.070$ is adequate for all values of f_{dy}. The minimum permissible value of q_d, applying Section 4.22 limitations, and recognizing that, for the wall sections considered,

$$q_d = \frac{p f_{dy}}{f'_{dc}} = \frac{\phi_c \, f_{dy}}{100 \, f'_{dc}} = \frac{\phi'_t \, f_{dy}}{200 \, f'_{dc}} = \frac{\phi_t \, f_{dy}}{180 \, f'_{dc}}$$

We obtain, by applying the $\phi_t \geqslant 0.50$ limitation of Section 4.22 the following expression for the minimum permissible value of q_d (wall).

$$q_d = \frac{0.50}{180} \times \frac{44,000}{2500} = 0.049.$$ Use $\phi_t = 0.56$ percent,

$f_{dy} = 44,000$ psi, $f'_{dc} = 2500$ psi, $D = 6$ in.

(d) <u>Eccentrically-Loaded Side Wall Cost Factor</u>

Concrete $$C_c = \left(\frac{D}{12}\right) X_c \qquad (3.35.33a)$$

$$C_c = \frac{6.00}{12} \times 1.025 = 0.51 \quad \$/\text{sq ft}$$

(X_c from Table 2.8)

Main Steel $$C_s = \frac{d}{12}\left(\frac{\phi'_t}{100}\right) X_s \qquad (3.35.34c)$$

$$C_s = \frac{0.9 \times 6.0}{12} \times 0.0056 \times 78.8 = 0.20 \quad \$/\text{sq ft} \quad (X_s \text{ from Table 2-7})$$

Temperature Steel
$$C_{st} = \frac{D}{12}\left(\frac{\phi_{te}}{100}\right) X_s \qquad (3.35.35)$$

$$C_{st} = \frac{6.00}{12} \times 0.001 \times 78.8 = 0.04 \quad \$/\text{sq ft}$$

Forms
$$C_f = X_f \qquad (3.35.36)$$

$$C_f = 1.00 \quad \$/\text{sq ft} \quad (X_f \text{ from Table 2-9})$$

Summary

C_c = 0.51

C_s = 0.20

C_{st} = 0.04

C_f = 1.00

C_t = 1.75 \$/sq ft

(e) End Wall Design

End walls can be designed either to resist an axial compressive load per foot, taken as some fraction of the compressive load on the side walls; or as a flexural member, fixed at the roof slab and at the foundation, which must resist the lateral component of the surface overpressure. This lateral component, which is normally assumed to vary between zero and the full value of the overpressure,[1, 3] will be taken in this study as 0.5 times the equivalent loading on the roof slab. However, the choice of analytical approaches is irrelevant in this particular instance, since the minimum dimension requirement of 6" for poured walls governs design. Where massive and rigid side walls or columns are employed, some reliance could also be given to arching theory[48, 49] in computing wall resistance to lateral loading. See Trial Design 4.34B for a detailed end wall design. For this problem, use same design for end walls as for side walls.

(f) End Wall Cost Factor

The cost factor for the end wall is identical with that for the side wall.

$$C_t = 1.75 \ \$/\text{sq ft}$$

(g) Interior Wall Design

The interior wall is designed to resist axial compression only. The concrete access arch is assumed to equal the cost/sq ft of the RCMU wall it replaces.

$$P = (12L + D)\, q$$

Try minimum wall size of D = 4.00 in. (see Section 4.22)

$$P = \left[(12 \times 7.0) + 4.00\right] \times 10 = 880 \text{ lb per lineal inch of wall}$$

Capacity of 4" RCMU = 4 x 1000 = 4000 lb per lineal inch of wall.

(Note that the term f'_{dc}, when used to indicate ultimate dynamic strength of reinforced masonry units, is used without the 0.85 reduction factor which is applied to axially-loaded, short reinforced concrete columns. Thus, $f'_{dc} = 1000$ psi is applied to the entire load-bearing area of a reinforced masonry unit.)

Use 4" RCMU.

(h) Interior Wall Cost Factor

$$C_t = X_{cm}$$

$$C_t = 1.01 \ \$/\text{sq ft} \qquad (X_{cm} \text{ from Table 2-10})$$

(i) Foundation Design

Alternatives are the use of a raft foundation, or of continuous wall footings with a minimum ground slab. Both will be investigated.

(1) Raft Foundation. Design is identical with that of the roof slab. D = 4.5 in.

(2) Continuous Footings. (See Section 3.39 for the analysis of continuous footings which support bearing walls. Note that the basic analysis assumes that there is no moment transfer from the wall to the footings.)

For exterior walls,

$$P = 72\, q\, L$$

$$P = 72 \times 10 \times 7 = 5040 \text{ lb/lineal foot of wall.}$$

Entering Table 3-59 with $\phi = 15^{o}$, c = 2000 lb/sq ft, and q = 10 psi, it is apparent that the minimum footing dimension as described in Section 4.22 will govern.

$$L_{footing} = 2.0 \text{ ft}$$

hence
$$\frac{P}{L} = \frac{5040}{2} = 2520 \text{ lb/ft/ft.}$$

and
$$\frac{D_{wall}}{L_{footing}} = \frac{6 \text{ in.}}{2 \text{ ft}} = 3$$

From Table 3-60, for P/L = 2520 and D/L = 3, with f'_c assumed at 2000 psi, linear extrapolation yields

$$\frac{d_{footing}}{L_{footing}} = 0.417$$

hence
$$d = 0.417 \times 2.0 = 0.83 \text{ in.}$$

Minimum dimension from Section 4.22 applies.

Use D = 8 in., d = 6 in.

The required percentage of main reinforcing steel, ϕ_c, can be obtained from Table 3-61 or Equation 3.39.6b. It is obvious that this percentage will be small, hence an unreinforced footing will probably be adequate if some nominal tensile strength is assigned to the uncracked concrete. Taking the tensile strength of plain concrete as $0.10\, f'_c$, which is considered to be acceptable for this application, the plain concrete footing is found to be adequate. This same footing design, since it is a minimum standard, is used for both side and end walls.

For interior walls,

$$P = 72 \times 10 \times 7 \times 2 = 10{,}080 \text{ lb/lineal foot of wall.}$$

Again, checking by use of Table 3-59, minimum dimensions govern. The required percentage of reinforcing steel can be computed from Equation 3.39.1 b. for P = 10,080 lb/ft, $d_{footing}$ = 6 in.and $L_{footing}$ = 2.0 ft.

$$\phi_c = \frac{P}{L}\left(\frac{0.09653}{f_{dy}}\right)\left[\frac{12L - D_{wall}}{d_{footing}}\right]^2$$

$$\phi_c = \frac{10,080}{2} \left(\frac{0.09653}{60,000}\right) \left[\frac{84 - 6}{6}\right]^2 = 1.37 \text{ percent}$$

For ground slab,

A 3-in.concrete ground slab with mesh reinforcement (A_s = 0.1 b D in each direction) will be assumed. This is a non-structural slab, and is not intended to resist blast loading. The mesh reinforcement is intended to control fragmentation of the slab. Since strength is secondary, use f'_c = 2000 psi.

(j) Foundation Cost Factors

(1) Raft Foundation. The in-place composite cost for the roof slab was found to be 1.66 \$/sq ft. Since the same design was assumed to be appropriate for the raft foundation (although it may not still be the least-cost design, since a somewhat different cost equation should be considered for the ground slab), it is simplest to identify the cost differences between the ground slab and the overhead slab. It is assumed that there is no form cost increase for the raft foundation due to depth.

Decrease in forming costs (see Equations 3.33.30e and 3.33.30f and Table 2-9)

$$\Delta C_f = k'_f - X_f = X_{f_1} + 0.012D - X_{f_2}$$

$$\Delta C_f = 0.88 + (0.012 \times 4.5) - 0.60 = 0.33 \text{ \$/sq ft.}$$

$$C_t \text{ for raft} = 1.66 - 0.33 = 1.33 \text{ \$/sq ft.}$$

(2) Continuous Footings

Exterior and interior wall footings

Concrete $$C_c = \left(\frac{D}{12}\right) X_c \qquad (3.39.9)$$

$$C_c = \frac{8.0}{12} \times 0.95 = 0.63 \text{ \$/sq ft}$$

Main Steel $$C_s = \frac{d}{12} \left(\frac{\phi_t}{100}\right) X_s$$

$$C_s = \frac{0.9 \times 4.5}{12} \times 0.0137 \times 78.8 = 0.36 \text{ \$/sq ft}$$

Temperature Steel $$C_{st} = \frac{D}{12} \left(\frac{\phi_{te}}{100}\right) X_s \qquad (3.39.11)$$

$$C_{st} = \frac{8.0}{12} \times 0.001 \times 78.8 = 0.05 \text{ \$/sq ft.}$$

Forms $$C_f = X_f \qquad (3.39.12)$$

$$C_f = 0.75 \text{ \$/sq ft.}$$

Summary for footings

$$C_c = 0.63$$
$$C_s = 0.36$$
$$C_{st} = 0.05$$
$$C_f = \underline{0.75}$$
$$C_t = 1.79 \text{ \$/sq ft.}$$

Floor slab

Concrete $$C_c = \left(\frac{D}{12}\right) X_c \qquad (3.39.9)$$

$$C_c = \frac{3}{12} \times 1.09 = 0.27 \text{ \$/sq ft}$$

Mesh reinforcement $$C_s = 2 \times \frac{3}{12} \times 0.001 \times 78.8 = 0.04 \text{ \$/sq ft}$$

Forms $$C_f = X_f \qquad (3.39.12)$$

$$C_f = 0.60 \text{ \$/sq ft}$$

Summary for floor slab

$$C_c = 0.27$$
$$C_s = 0.04$$
$$C_{st} = 0.00$$
$$C_f = 0.60$$
$$C_t = 0.91 \text{ \$/sq ft}$$

Side and end wall extensions

It is assumed that the tops of all footings will be located one foot below the top of the floor slab. Thus, if a continuous-footing foundation is used, the side and end walls must be extended one additional foot in depth. The cost factors for these foundation walls are the same as derived earlier.

Side wall	C_t = 1.75 \$/sq ft
End wall	C_t = 1.71 \$/sq ft
Interior wall	C_t = 1.01 \$/sq ft

Footing excavation

The additional cost of excavating for footings must enter into a cost comparison between a raft foundation and continuous footings. Assume that foundation wall extensions and the continuous footings require the excavation of a 3-ft width trench extending down to the bottom of the footing. (Conceivably, in firm soil, a 2-ft width of unsupported trench could form the side forms for the footing. However, this possible refinement will not be included in the analysis).

For estimating purposes, take

depth of footing trench = 1.57 ft
width of footing trench = 3.0 ft
volume of trench excavation = 5.0 cu ft per lineal ft of trench

Assuming a unit cost of 0.10 \$/cu ft for trench excavation, the cost factor per lineal foot of trench is

$$C_t = 5.0 \times 0.10 = 0.50 \text{ \$/lineal ft}$$

(3) Cost Comparison For Foundation Alternatives

Raft foundation. Required dimensions (see Figure 4-2) are 60.0 ft x 15.5 ft.

$$C_t = 60.0 \times 15.5 \times 1.33 = \$1237$$

Continuous footing system

Footings

length = (3 x 59.0) + (2 x 15.5) = 208 lineal ft
area = 2.0 x 208 = 416 sq ft
C_t = 416 x 1.79 = \$745

Side wall extensions

length = 2 x 60.0 = 120 lineal ft
area = 1 x 120 = 120 sq ft
C_t = 120 x 1.75 = \$210

End wall extensions

$$\text{length} = 2 \times 15.5 = 31.0 \text{ lineal ft}$$

$$\text{area} = 1 \times 31.0 = 31.0 \text{ sq ft}$$

$$C_t = 31.0 \times 1.71 = \$53$$

Interior wall extension

$$\text{length} = 59.0 \text{ lineal ft}$$

$$\text{area} = 1 \times 59.0 = 59.0 \text{ sq ft}$$

$$C_t = 59.0 \times 1.01 = \$60$$

Floor slab

Required dimensions are 59.0 x 14.1 ft

$$C_t = 59.0 \times 14.1 \times 0.91 = \$\ 757$$

Footing excavation

$$\text{length} = 2(60 + 15.5) + 59.0 = 210 \text{ lineal ft}$$

$$C_t = 210 \times 0.50 = \$105$$

Summary

$$\sum C_t = \$1930$$

Use raft foundation.

(k) Required Excavation

1) Minimum cover $h = 3.5$ ft

2) Cover required for full burial $h = 0.0$ ft

3) Radiation burial requirement

$$d_e = 3.5 + 0.020q \qquad (4.21.1b)$$

$$d_e = 3.5 + (0.020 \times 10) = 3.70 \text{ ft}$$

For shielding, 1.0 ft of concrete $\approx$ 1.5 ft of earth

$$3.70 - (1.5 \times D/12) = 3.70 - (1.5 \times 4.50/12) = 3.14 \text{ ft}$$

$3.14 < 3.50$

Use 3.50 ft cover

Total Depth of Excavation (see Fig. 4-2)

$$z = h + H + \frac{D_{roof} + D_{floor}}{12} \qquad (4.34.1)$$

$$z = 3.5 + 8.0 + \frac{4.5 + 4.5}{12} = 12.25 \text{ ft}$$

Following the criteria presented in Section 4.24,

$$\text{Volume} = \frac{z}{2}(A_1 + A_2) \qquad (4.24.1)$$

$$\text{Volume} = \frac{12.25}{2}\left[(60.0 \times 15.5) + (84.5 \times 40.0)\right] \approx 26,400 \text{ cu ft}$$

Note: The length and width of A_2 are related to the dimensions of A_1 by adding the quantity 2 z to both dimensions.

$$\text{Cubicle Gross Volume} = 15.5 \times 60.0 \times \left[8.0 + \left(\frac{4.5 + 4.5}{12}\right)\right] = 8140 \text{ cu ft.}$$

(l) Entrance Way

Entrance way costs for the cubicle are found in Table 4-4.

$$C_T = \$2750$$

(m) Total Cost

Roof Slab	1.66 x 15.5 x 60.0 =	1544
Foundation	(see section k of this design)	1237
Side Wall	1.75 x 59.0 x 8.0 x 2 =	1650
End Wall	1.75 x 15.5 x 8.0 x 2 =	435
Interior Wall	1.01 x 59.0 x 8.0 =	477
Excavation	0.036 x 26,400 =	950
Back Fill	0.033 x (26,400 - 8140) =	603
Haul	0.026 x 8140 =	212
Entrance Way		2750
	Total	$9858

(n) Cubicle Net Floor Area

Net area = (14.5 x 59.0) - (59 x 0.33) = 835 sq ft

(o) Cubicle Net Volume

Net volume = 835 x 8.0 = 6680 cu ft

Table 4-4
COST FACTORS FOR SHELTER ENTRANCE WAY

Type of Shelter	Equivalent Pressure (psi)	A[1] ($/lin ft)	B[2] ($/lin ft)	Entrance Way Length (ft.)	Total[3] Cost ($)
One Story Cubicle	10	40	10	49	2750
	25	40	10	49	2750
	50	46	11	49	3090
	75	51	12	50	3450
	100	56	12	51	3765
	150	67	13	52	4460
	200	78	13	53	5120
	250	83	14	54	5540
	325	89	15	55	6020
One Story Cylinder	25	40	15	51	3100
	50	46	15	51	3410
	75	51	16	52	3780
	100	56	17	53	4110
	150	67	18	54	4890
	200	78	19	55	5630
	250	83	20	56	6070
	325	89	21	57	6570
Two Story Cylinder	25	40	15	50	3950
	50	46	15	50	3360
	75	51	16	51	3730
	100	56	17	52	4050
	150	67	18	53	4820
	200	78	19	54	5550
	250	83	20	55	5990
	325	89	21	56	6480
One Story Arch	15	40	11	50	2850
	25	40	11	50	2850
	50	46	11	50	3150
	75	51	12	51	3510
	100	56	13	52	3890
	150	67	14	53	4590
Three Story Sphere	25	40	17	51	3200
	50	46	17	51	3510
	75	51	18	52	3820
	100	56	19	53	4280
	150	67	20	54	5000
	200	78	21	55	5740
	250	83	22	56	6180
	325	89	23	57	6690
One Story Dome	25	40	25	62	4303
	50	46	25	62	4700
	75	51	26	63	5150
	100	56	27	64	5620
	150	67	28	65	6470
Two Story Dome	25	40	16	50	3100
	50	46	16	50	3400
	75	51	17	51	3770
	100	56	18	52	4150
	150	67	19	53	4860

Note: (1) Column A Structural Cost Factor for Entrance Way
(2) Column B Excavation Cost Factor for Entrance Way
(3) Total Cost Includes $300.00 per Entrance Way for Stair Costs

TRIAL DESIGN 4.34B

CONFIGURATION:

One story cubicle (see Figure 4-3).

STRUCTURAL SYSTEM:

Monolithic, one-way slab-style A-double with one interior partition. (See Table 4-3)

DESIGN PARAMETERS:

q = 25 psi equivalent pressure, including weight of slab and earth cover

L = 14.0 ft clear span

(a) Roof Slab Design

From Table 3-15, for L = 14.0 ft and q = 25 psi, the minimum-cost structural parameters for the overhead one-way slab with fixed-end support are.

$$f_{dy} = 75,000 \text{ psi}$$

$$f'_c = 4100 \text{ psi}$$

$$\phi_c = 1.39 \text{ percent}$$

$$\phi_v = 0.50 \text{ percent}$$

$$D = 7.6 \text{ in.}$$

Check section for "pure" shear. Ultimate shear capacity of gross section is

$$V_u = 0.22\ b\ d\ f'_c$$

$$V_u = 0.22 \times 1.0 \times 0.9 \times 7.6 \times 4100 = 6170 \text{ lb/in width}$$

Maximum shear in slab, which is assumed to occur at the supports, is

$$V = 6\ q\ L$$

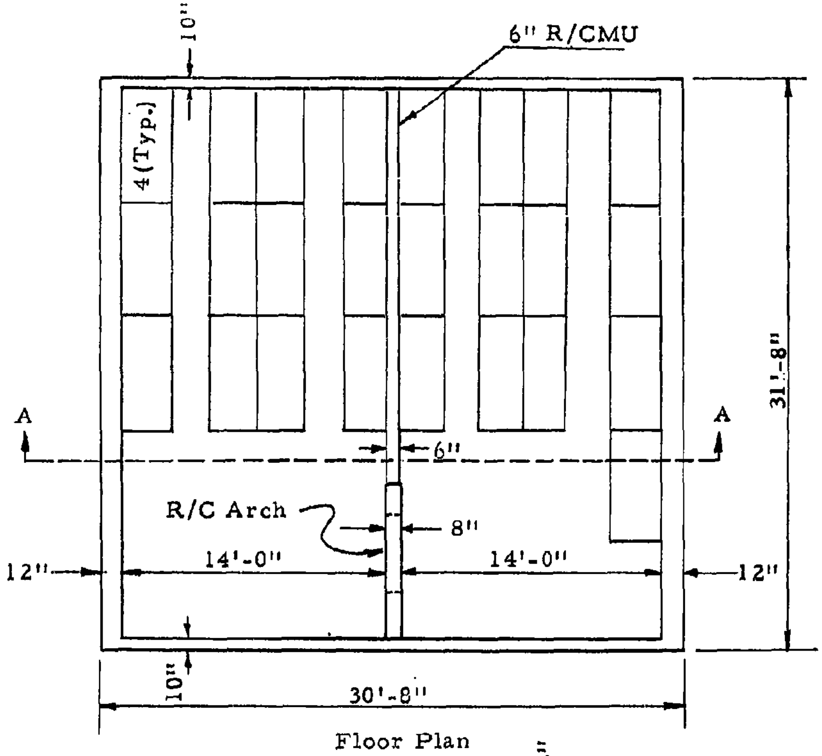
$$V = 6 \times 25 \times 14 = 2100 \text{ lb/in width}$$

$2100 < 6170$ Section is adequate.

(b) Roof Slab Cost Factor

From Table 3-15

$$C_t = \$3.09/\text{sq ft}$$



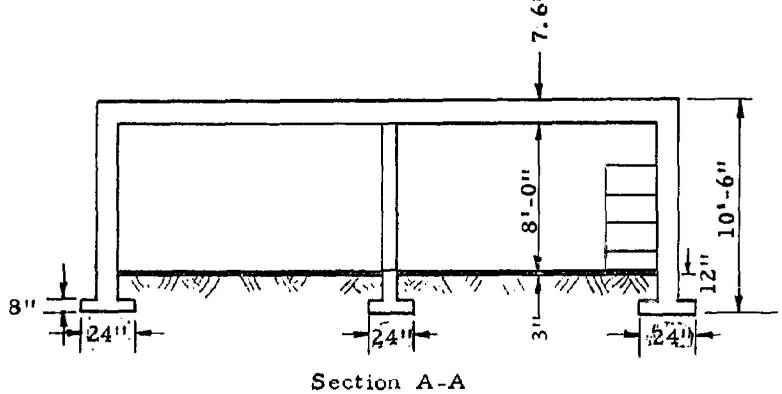


Figure 4-3

SHELTER LAYOUT TRIAL DESIGN 4.34B

(c) Eccentrically-Loaded Side Wall Design

Following the guide lines presented in Section 3.35(4), take f'_{dc} = 2500 psi, f_{dy} = 60,000 psi. From Section 4.22, the minimum permissible value of ϕ'_t is 0.50 and the minimum permissible value of D is 6 in. The question of whether or not these minimum values correspond with minimum wall cost must be explored by analyzing trial solutions for the wall.

Required for use with Table 3-41 to 3-44, inclusive

$$\frac{q}{f'_{dc}} = \frac{25}{2500} \cong 0.010$$

$$q_d \text{ (wall)} = \frac{\phi'_t}{200} \quad \frac{f_{dy}}{f'_{dc}} = 0.12\,\phi'_t \text{ for } f_{dy} = 60,000 \text{ psi and } f'_{dc} = 2500 \text{ psi}$$

The minimum permissible wall reinforcement from Section 4-22, is ϕ_t = 0.50 percent. For d = 0.9D, $\phi_t = 0.9\,\phi'_t$. Substituting, we obtain $q_d \geq 0.067$ as a minimum requirement for main reinforcing steel in the wall. Also, again applying Section 4-22, we can state that $\frac{d_{wall}}{L_{slab}}$ must be at least equal to $\frac{6}{14}$ = 0.43. Using Table 3-43, designs for the wall which satisfy the requirement $\frac{q}{f'_{dc}}$ = 0.01 for f_{dy} = 60,000 psi and f'_{dc} = 2500 psi will include the following,

1) $q_d = 0.180$ $\qquad \frac{d_{wall}}{L_{slab}} = 0.75$

hence

ϕ'_t = 1.50 percent

d = 0.75 x 14 = 10.5 in.

$D = \frac{d}{0.9} = \frac{10.5}{0.9} \approx 11.75$ in.

2) $q_d = 0.100$ $\qquad$ d/L = 1.00

hence

ϕ'_t = 0.835 percent

d = 1.00 x 14 = 14.0 in.

$D = \frac{14}{0.9} \approx 15.5$ in.

3) $q_d = 0.067$ $d/L = 1.25$

hence $\phi'_t = 0.55$ percent

$$d = 1.25 \times 14 = 17.5 \text{ in.}$$

$$D = \frac{17.5}{0.9} \approx 19.5 \text{ in.}$$

(d) Eccentrically-Loaded Side Wall Cost Factor

The design alternatives just described can be compared on the basis of in-place costs.

Trial No. 1

Concrete $C_c = \left(\frac{D}{12}\right) X_c$ (Recall $f'_{dc} = 1.25 f'_c$)

$$C_c = \frac{11.75}{12} \times 1.00 = 0.98 \text{ \$/sq ft}$$

Main Steel $C_s = \frac{d}{12}\left(\frac{\phi'_{t'}}{100}\right) X_s$

$$C_s = \frac{10.5}{12} \times \frac{1.50}{100} \times 78.8 = 1.04 \text{ \$/sq ft}$$

Temperature Steel $C_{st} = \frac{D}{12}\left(\frac{\phi_{te}}{100}\right) X_s$

$$C_{st} = \frac{11.75}{12} \times \frac{0.1}{100} \times 78.8 = 0.08 \text{ \$/sq ft}$$

Forms $C_f = X_f$

$$C_f = 1.00 \text{ \$/sq ft}$$

Summary for Trial No. 1

$$C_c = 0.98$$
$$C_s = 1.04$$
$$C_{st} = 0.08$$
$$C_f = \underline{1.00}$$
$$C_t = 3.10 \text{ \$/sq ft}$$

Trial No. 2

Concrete $C_c = \frac{15.5}{12} \times 1.00 = 1.29 \text{ \$/sq ft}$

Main Steel $C_s = \frac{14.0}{12} \times \frac{0.835}{100} \times 78.8 = 0.77$ \$/sq ft

Temperature Steel $C_{st} = \frac{15.5}{12} \times \frac{0.10}{100} \times 78.8 = 0.10$ \$/sq ft

Forms $C_f = 1.00$ \$/sq ft

Summary for Trial No. 2

$C_c = 1.29$

$C_s = 0.77$

$C_{st} = 0.10$

$C_f = \underline{1.00}$

$C_t = 3.16$ \$/sq ft

Trial No. 3

Concrete $C_c = \frac{19.5}{12} \times 1.00 = 1.63$ \$/sq ft

Main Steel $C_s = \frac{17.5}{12} \times \frac{0.55}{100} \times 78.8 = 0.63$ \$/sq ft

Temperature Steel $C_{st} = \frac{19.5}{12} \times \frac{0.10}{100} \times 78.8 = 0.13$ \$/sq ft

Forms $C_f = 1.00$ \$/sq ft

Summary for Trial No. 3

$C_c = 1.63$

$C_s = 0.63$

$C_{st} = 0.13$

$C_f = \underline{1.00}$

$C_t = 3.39$ \$/sq ft

It is apparent that, of the three trial solutions, the one based on the near-minimum value for total wall thickness, D, results in the least in-place cost for the wall. This economic advantage would be enhanced if excavation costs were included in the cost comparisons. There still remains a question as to whether or not other values of f_{dy} and f'_c would result in lower wall

costs. However, an examination of Tables 3-41 to 3-44, inclusive, indicates that q_d (wall) is essentially constant, within the $\frac{d_{wall}}{L_{slab}}$ range of primary interest, for given values of d/L and $\frac{q}{f'_{dc}}$. We can then state, as an approximation,

$$q_d\,(\text{wall}) \propto \frac{d}{L}, \text{ for fixed } \frac{q}{f'_{dc}} \text{ and all } f_{dy}$$

since $$q_d = \frac{\phi'_t\, f_{dy}}{200\, f'_{dc}} = \frac{A_s}{d}\,\frac{f_{dy}}{f'_{dc}}, \text{ we can also write}$$

$$\frac{A_s\, f_{dy}}{f'_{dc}} \propto \frac{d^2}{L}, \text{ for fixed } \frac{q}{f'_{dc}} \text{ and all } f_{dy}$$

The cost of the reinforcing steel is directly proportional to $A_s\, f_{dy}$ and, for the in-place costs of reinforcing steel listed in Table 2-7, f_{dy} = 60,000 psi is an optimum choice. A limited number of trial solutions have suggested that f_{dy} = 60,000 psi and f'_{dc} = 2500 psi (f'_c = 2000 psi) are associated with minimum costs. This general relationship, conceivably, could be altered by the minimum requirements for ϕ_t and D.

Use D = 12 in., C_t = 3.10 \$/sq ft

(e) <u>End Wall Design</u>

Assume that the lateral component of overpressure on the wall is one-half that acting on the roof. Design the end walls as beams spanning between the roof and the floor slab or foundation. Two cases must therefore be considered.

(1) Raft Foundation. End wall can be represented as a one-way reinforced concrete beam whose clear span length is L = H = 8.0 ft and whose unit uniform loading is $\frac{q}{2} = \frac{25}{2}$ = 12.5 psi. Consider the beam as fully-restrained at the floor and roof slabs. Table 3-15, which supplies structural parameters and in-place cost data for minimum-cost one-way reinforced concrete slabs, does not include q = 12.5 psi and L = 8.0 ft. However, for q = 10 psi and L equal to either 7.0 ft or 10.5 ft, the applicable structural parameters for the minimum cost slabs are

f_{dy} = 75,000 psi or f_{dy} = 75,000 psi

$f'_c = 3600$ psi or $f'_c = 6000$ psi

$\phi_v = 0.0$ $\phi_v = 0.0$

These relationships suggest that $f_{dy} = 75,000$ psi, $f'_c = 4500$ psi, and $\phi_v = 0$ should approach the minimum-cost structural parameters for the end wall. Entering Table 3-13 with these values, and assuming $\theta' = 0.25$, we obtain the following.

$$\phi_c = \frac{f'_c}{k} = \frac{4500}{9140} = 0.49 \text{ percent}$$

$$\frac{qL^2}{\phi_c} = \frac{12.5 \times (8)^2}{0.49} = 1630$$

Since the resistance functions of Table 3-13 do not include values as low as 1630, it is necessary to compute the slab depth from the basic equations.

$$d = \sqrt{\frac{1000\, q_f\, L^2}{\phi_c\, f_{dy}}} \qquad (3.33.21)$$

$$d = \sqrt{\frac{1000 \times 12.5 \times 64}{0.49 \times 75,000}} = 4.7 \text{ in.}$$

Use D = 6.0 in., d = 4.7 in. and $\phi_c = 0.49$ percent

(2) Continuous Footings and Ground Slab. For this case, assuming that the top of the footing is placed 1'-0" below the top of the ground slab, the end wall must span a clear distance of 9'-0" between the roof slab and the continuous footing. It is reasonable to assume that the end wall, which will be designed as a one-way reinforced concrete slab, is hinged at the footing level and fully restrained at the roof level.

Table 3-15, which supplies minimum-cost data for fixed-end overhead slabs, is of little assistance in determining minimum-cost structural parameters for end walls which have one end fixed and one end fully restrained. Since diagonal tension adjacent to the hinged end may constitute the controlling design condition, it should be advantageous to use a high value for f'_c. We

will assume, without further investigation, that the minimum-cost structural parameters for this design situation can also be represented by,

$$f_{dy} = 75,000 \text{ psi}$$

$$f'_c = 4500 \text{ psi}$$

$$\phi_v = 0.0$$

From Table 3-14, for q = 12.5 psi and L = 9.0 ft, we obtain the following for $\theta' = 0.25$,

$$\phi_c = \frac{f'_c}{k} = \frac{4500}{17660} = 0.25 \text{ percent}$$

$$\frac{q\,L^2}{\phi_c} = \frac{12.5 \times (9)^2}{0.25} = 4050$$

$$d = \sqrt{\frac{1333\, q_f L^2}{\phi_c\, f_{dy}}} \qquad \text{(from 3.33.15)}$$

$$d = \sqrt{\frac{1333 \times 12.5 \times 9^2}{0.25 \times 75,000}} = 8.5 \text{ in.}$$

Use D = 10.0 in., d = 8.5 in. and ϕ_c = 0.25 percent

(f) End Wall Cost Factor

Both design alternatives will be examined.

(1) Raft Foundation Alternatives

Concrete
$$C_c = \left(\frac{D}{12}\right) X_c \qquad (3.33.30a)$$

$$C_c = \frac{6}{12} \times 1.25 = 0.63 \text{ \$/sq ft}$$

Main Steel
$$C_s = \frac{X_s\, \phi_c\, d}{1200} \left[1.33 + \frac{0.278}{L} \frac{f_{dy}}{f'_c}\right] \qquad (3.33.30b)$$

$$C_s = \left(\frac{85.8 \times 0.49 \times 4.7}{1200}\right) \left[1.33 + \left(\frac{0.278}{8} \times \frac{75,000}{4500}\right)\right] = 0.32 \text{ \$/sq ft}$$

Temperature Steel
$$C_{st} = \frac{D}{12} \left(\frac{\phi_{te}}{100}\right) X_s \qquad (3.35.35)$$

$$C_{st} = \frac{6}{12} \times \frac{0.1}{100} \times 78.8 = 0.04 \text{ \$/sq ft}$$

Forms $C_f = X_f$

$C_f = 1.00$ \$/sq ft (3.35, 36)

Summary for Raft Foundation Alternatives

$C_c = 0.63$

$C_s = 0.32$

$C_{st} = 0.04$

$C_f = \underline{1.00}$

$C_t = 1.99$ \$/sq ft (see Figure 4-3)

$C_t = 2 \times 30.67 \times 8.0 \times 1.99 = \$976$

(2) Continuous Footings and Ground Slab Alternative

Concrete $C_c = \frac{10}{12} \times 1.25 = 1.04$ \$/sq ft

Main Steel — Assume that cost of main steel for the one end fixed, one end hinged slab, with $\theta' = 0.25$, is the same as for the fixed-fixed slab. This should slightly over-state the probably cost of such steel.

$$C_s = \left(\frac{85.8 \times 0.25 \times 8.5}{1200}\right) \left[1.33 + \left(\frac{0.278}{9} \times \frac{75,000}{4500}\right)\right] = 0.28 \text{ \$/sq ft}$$

Temperature Steel $C_{st} = \frac{10}{12} \times \frac{0.1}{100} \times 78.8 = 0.07$ \$/sq ft

Forms $C_f = 1.00$ \$/sq ft

Summary

$C_c = 1.04$

$C_s = 0.28$

$C_{st} = 0.07$

$C_f = \underline{1.00}$

$C_t = 2.39$ \$/sq ft

$C_t = 2 \times 30.67 \times 9.0 \times 2.39 = \$1319$

The final decision between the two types of end walls will involve an economic comparison between the foundation alternatives, giving recognition to the costs derived for the end walls.

(g) Interior Wall Design

$$P = (12L + D)\, q$$

Assume 6 in. interior wall.

$$P = \left[(12 \times 14.0) + 6.0\right] 25 = 4350 \text{ lb/lineal in. of wall}$$

Use 6 in. RCMU (allowable = 6000 lb/lineal in. of wall).

(h) Interior Wall Cost Factor

For 6 in. RCMU

$$C_t = X_{cm}$$

$$C_t = 1.10 \text{ \$/sq ft}$$

(i) Foundation Design

Both ground slab and continuous footings are investigated, using a similar procedure to that described for Design Problem 4-34 A.

(1) Raft Foundation. Design is identical with that of the roof slab. Use D = 7.6 in.

(2) Continuous Footings and Ground Slab. See Section 3.39 for analysis and outline.

For eccentrically-loaded side walls,

$$P = 72\, q\, L$$

$$P = 72 \times 25 \times 14 = 25{,}200 \text{ lb/lineal ft of wall}$$

From Table 3-59 for $\phi = 15°$, $c = 2000$ lb/sq ft and $q = 25$ psi, the ultimate bearing capacity of the soil for a footing less than 4 ft wide is 24 kips/sq ft.

Use $L_{footing} = 2.0$ ft (minimum dimension governs-Section 4.22)

Ultimate capacity of the footing.

$$P = 2.0 \times 24 = 48 \text{ kips/lineal ft of wall}$$

Required for use with Table 3-60.

$$P/L = \frac{25.2}{2.0} = 12.6 \text{ kips}$$

$$\frac{D_{wall}}{L_{footing}} = \frac{12.00 \text{ in.}}{2.0 \text{ ft}} = 6$$

From Table 3-60 for $f'_c = 2000$ psi, $\frac{P}{L} = 12,600$ lb and $\frac{D}{L} = 6$.

$$\frac{d_{footing}}{L_{footing}} = 0.92 \qquad d = 0.92 \times 2.0 = 1.84 \text{ in.}$$

$L = 2.0$ ft $\qquad D = 8.00$ in. (min. dim. governs - Section 4.22)

Use $d = 6.00$ in.

A minimum concrete cover of 2 in. is specified for footings to allow for uneven ground conditions in the bottom of the footing trench.

Required for use with Table 3-61

$$\frac{d_{footing}}{12L_{footing} - D_{wall}} = \frac{6.00}{24.0 - 12.00} = 0.50$$

From Table 3-61 for $\frac{P}{L} = 12,600$, $\frac{d}{12L - D} = 0.50$ and $f_{dy} = 60,000$ psi

$$\phi_c = 0.10$$

A plain concrete footing with D = 8 in. and the ultimate tensile strength of the concrete taken equal to 10 percent of f'_c if found to be adequate in this instance.

It should be noted that Tables 3-60 and 3-61 are based on a shearing-flexure mode of failure. The possibility still exists for a premature diagonal tension mode of failure, although normally this does not govern design.

Check for diagonal tension.

$$\phi_c = \frac{1,930,000 f'_c}{f^2_{dy}} \tag{3.39.4}$$

$$\phi_c = \frac{1.93 \times 10^6 \times 2 \times 10^3}{36 \times 10^8} = 1.07$$

Actual $\phi_c = 0.00 < 1.07$ O.K.

<u>For end walls,</u>

The same footing is used for both eccentrically-loaded side walls and end walls.

For interior walls,

$$P = 144\, q\, L$$

$$P = 144 \times 25 \times 14 = 50.4 \text{ kips/lineal ft of wall}$$

Assume L = 2.0 ft

$$\frac{P}{L} = \frac{50.4}{2.0} = 25.2 \text{ kips/sq ft}$$

Based on Table 3-59, the ultimate allowable load on a 2.0 ft footing is 24.0 kips/sq ft.

Allowing this slight under-design

$$L = 2.0 \text{ ft}$$

Required for use with Table 3-60

$$\frac{D_{wall}}{L_{footing}} = \frac{6.00 \text{ in.}}{2.0 \text{ ft}} = 3$$

From Table 3-60, for f'_c = 2000 psi, $\frac{P}{L}$ = 25,000 lb and $\frac{D}{L}$ = 3.

$$\frac{d_{footing}}{L_{footing}} = 2.95 \qquad d = 2.0 \times 2.95 = 5.90$$

$$+\ \underline{2.00} \text{ cover}$$

L = 2.0 ft, Total 7.90 in.

Use D = 8.00 in.

Required for use with Table 3-61

$$\frac{d_{footing}}{12\, L_{footing} - D_{wall}} = \frac{5.90}{24.0 - 6.00} = 0.33$$

From Table 3-61, for $\frac{P}{L}$ = 25,000, $\frac{d}{12L - D}$ = 0.33 and f_{dy} = 75,000 psi

$$\phi_c = 0.29 \text{ percent}$$

Ground slab

Use 3 in. ground slab, f'_c = 2000 psi, with $A_s = 0.10\, b\, D$ in each direction. (Mesh reinforcement, see Design Problem 4-34A).

(j) Foundation Cost Factors

In computing costs of foundation which employ continuous footings,

it is assumed that the top of the footing will rest at one foot below the top of the floor slab. All walls will be extended to the footing elevation.

(1) Raft Foundation. In-place cost of equivalent overhead slab (see "Roof Slab Design" for this design problem) is 3.09 \$/sq ft. Computing the decrease in cost for a raft foundation of comparable strength (see Design Problem 4-34A, Section j), we obtain the following.

Decrease in forming costs

$$\Delta C_f = k'_f - X_f = X_{f_1} + 0.012D - X_{f_2}$$

$$= 0.88 + (0.012 \times 7.6) - 0.60 = 0.37 \text{ \$/sq ft}$$

C_t for raft = 3.09 - 0.37 = 2.72 \$/sq ft

(2) Continuous Footings and Ground Slab

<u>Exterior wall footings</u>

Concrete $$C_c = \left(\frac{D}{12}\right) X_c \qquad (3.39.9)$$

$$C_c = \frac{8.00}{12} \times 0.95 = 0.64 \text{ \$/sq ft}$$

Main Steel None required.

Temperature Steel $$C_{st} = \frac{D}{12}\left(\frac{\phi_{te}}{100}\right) X_s \qquad (3.39.11)$$

$$C_{st} = \frac{8.00}{12} \times 0.001 \times 78.8 = 0.05 \text{ \$/sq ft}$$

Forms $$C_f = X_f \qquad (3.39.12)$$

$$C_f = 0.75 \text{ \$/sq ft}$$

Summary

$$C_c = 0.64$$

$$C_s = 0.00$$

$$C_{st} = 0.05$$

$$C_f = \underline{0.75}$$

$$C_t = 1.44 \text{ \$/sq ft}$$

<u>Interior wall footings</u>

Concrete $$C_c = \left(\frac{D}{12}\right) X_c \qquad (3.39.9)$$

$$C_c = \frac{8.00}{12} \times 0.95 = 0.64 \ \$/\text{sq ft}$$

Main Steel $$C_s = \frac{d}{12}\left(\frac{\phi_c}{100}\right) X_s \qquad (3.39.10a)$$

$$C_s = \frac{5.90}{12} \times 0.0029 \times 85.8 = 0.12 \ \$/\text{sq ft}$$

Temperature Steel $$C_{st} = \frac{D}{12}\left(\frac{\phi_{te}}{100}\right) X_s \qquad (3.39.11)$$

$$C_{st} = \frac{8.00}{12} \times 0.01 \times 78.8 = 0.05 \ \$/\text{sq ft}$$

Forms $$C_f = X_f \qquad (3.39.12)$$

$$C_f = 0.75 \ \$/\text{sq ft}$$

Summary

$$C_c = 0.64$$
$$C_s = 0.12$$
$$C_{st} = 0.05$$
$$C_f = \underline{0.75}$$
$$C_t = 1.56 \ \$/\text{sq ft}$$

Floor slab

Concrete $$C_c = \left(\frac{D}{12}\right) X_c \qquad (3.33.30a)$$

$$C_c = \frac{3.0}{12} \times 1.09 = 0.27 \ \$/\text{sq ft}$$

Mesh Reinforcement

$$C_s = 2 \times \frac{3.0}{12} \times \frac{0.10}{100} \times 78.8 = 0.04 \ \$/\text{sq ft}$$

Forms $$C_f = X_f$$

$$C_f = 0.60 \ \$/\text{sq ft}$$

Summary

$$C_c = 0.27$$
$$C_s = 0.04$$
$$C_f = \underline{0.60}$$
$$C_t = 0.91 \ \$/\text{sq ft}$$

Wall extensions

From Sections (d), (f), and (h) of this study, the following cost factors are available.

Side walls	C_t = 3.10 \$/sq ft
Interior walls	C_t = 1.10 \$/sq ft
End walls	Evaluate on the basis of total in-place costs, since design of end wall differs for the two types of foundation. For the isolated-footing type, the difference in total end wall cost is C_t = \$1319 - \$976 = \$343

Footing excavation (see Design Problem 4-34A).

Using the same assumptions as in the preceeding problem, and correcting for the applicable lengths and depths of continuous footings we obtain the following.

depth of footing trench = 1.67 ft
width of footing trench = 3.0 ft
volume of trench excavation = 5.0 cu ft per lineal ft of trench
Assuming a unit cost of 0.10 \$/cu ft for trench excavation,

$$C_t = 5.0 \times 0.10 = 0.50 \text{ \$/lineal ft}$$

(3) Cost Comparison For Foundation Alternatives

Raft Foundation. Required dimensions (see Figure 4-3) are approximately 30.67 ft x 31.67 ft.

$$C_t = 30.67 \times 31.67 \times 2.72 = \$2640$$

Continuous Footing System

footings(exterior) length = 2(31.67 + 28.67) = 120.7 lineal ft
area = 2.0 x 120.7 = 241.4 sq ft
C_t = 241.4 x 1.45 = \$350

footings(interior) length = 30.0 lineal ft

area = 2.0 x 30.0 = 60.0 sq ft

C_t = 60.0 x 1.56 = \$94

side wall extensions. (see Section (d) of this design example)

length = 2 x 31.67 = 63.3 lineal ft

area = 1 x 63.3 = 63.3 sq ft

C_t = 63.3 x 3.10 = \$196

end wall extensions (seeSection (f) of this design example)

ΔC_t = \$343

interior wall extensions

length = 30.0 lineal ft

area = 1 x 30.0 = 30.0 sq ft

C_t = 30.0 x 1.10 = \$33

footing excavation

length = 2(31.67 + 28.67) + 30 = 150.7 lineal ft

C_t = 150.7 x 0.50 = \$75

floor slab Required dimensions are 28.17 ft x 30.0 ft

C_t = 28.17 x 30.0 x 0.91 = \$770

Summary $\sum C_t$ = \$1861 < \$2640

Use continuous footing system, with D = 10.0 in. for end walls.

(k) Required Excavation

1) Minimum cover h = 3.5 ft
2) Cover required for full burial h = 0.0 ft
3) Radiation burial requirement

$$d_e = 3.5 + 0.020\,q \qquad (4.21.1b)$$

$$d_e = 3.5 + (0.020 \times 25) = 4.00 \text{ ft}$$

For shielding, 1.0 ft of concrete ≈ 1.5 ft of earth

$$4.0 - (1.5 \times D/12) = 4.0 - (1.5 \times 7.6/12) \quad 3.05 \text{ ft}$$

$$3.05 < 3.50$$

Use 3.50 ft cover

Total depth of excavation (see Figure 4-3, trenching for footings excluded)

$$z = h + H + \left(\frac{D_{roof} + D_{floor}}{12}\right) \tag{4.34.1}$$

$$z = 3.5 + 8.0 + \left(\frac{7.6 + 3.0}{12}\right) = 12.3 \text{ ft}$$

Following the criteria presented in Section 4.24,

$$\text{Volume} = \frac{z}{2} \; (A_1 + A_2) \tag{4.24.1}$$

$$\text{Volume} = \frac{12.3}{2} \left[(31.67 \times 30.67) + (56.3 \times 55.3)\right] = 25,100 \text{ cu ft}$$

Cubicle gross volume

$$\text{Volume} = L_T B_T \left[H + \frac{\left(D_{roof} + D_{floor}\right)}{12}\right]$$

$$\text{Volume} = 31.67 \times 30.67 \times \left[8.00 + \left(\frac{7.6 + 3.0}{12}\right)\right] = 8620 \text{ cu ft}$$

(l) Entrance Way

From Table 4-4

$$C_T = \$2750$$

(m) Total Cost

Roof Slab	3.09 x 30.67 x 31.67 =	3000
Side Wall	3.10 x 31.67 x 9.0 x 2 =	1770
End Wall	2.39 x 30.67 x 9.0 x 2 =	1319
Interior Wall	1.10 x 30.0 x 9.0 =	297
Floor Slab	0.91 x 28.17 x 30.00 =	770
Side Wall Footing	1.45 x 31.67 x 2.0 x 2 =	184
End Wall Footing	1.45 x 28.67 x 2.0 x 2 =	167
Interior Wall Footing	1.58 x 30.00 x 2.0 =	95
Footing Trench	0.50 x 150.6 =	75

Excavation	0.036 x 25,100 =	905
Backfill	0.033 x (25,100 - 8620) =	544
Haul	0.026 x 8620 =	224
Entrance Way		2750
	Total	$12,100

(n) Cubicle Net Floor Area

Net area = 2 x 14.0 x 30.0 = 840 sq ft

(o) Cubicle Net Volume

Net volume = 840 x 8.0 = 6720 cu ft

TRIAL DESIGN 4.34C

CONFIGURATION:

One story cubicle (see Figure 4-4)

STRUCTURAL SYSTEM:

Monolithic two-way slab, isotropic-style A-quad. (see Table 4-3)

DESIGN PARAMETERS:

q = 10 psi equivalent pressure, including weight of slab and earth cover

L = 28.0 ft

$$\alpha = \frac{28.0}{30.0} = 0.933$$

(a) Roof Slab Design

From Table 3-39, for L = 28.0 ft, α = 0.9 and q = 10 psi, the minimum-cost structural parameters for the overhead isotropic two-way slab with fixed-edge support and no web reinforcement are,

f_{dy} = 75,000 psi

f'_c = 2000 psi

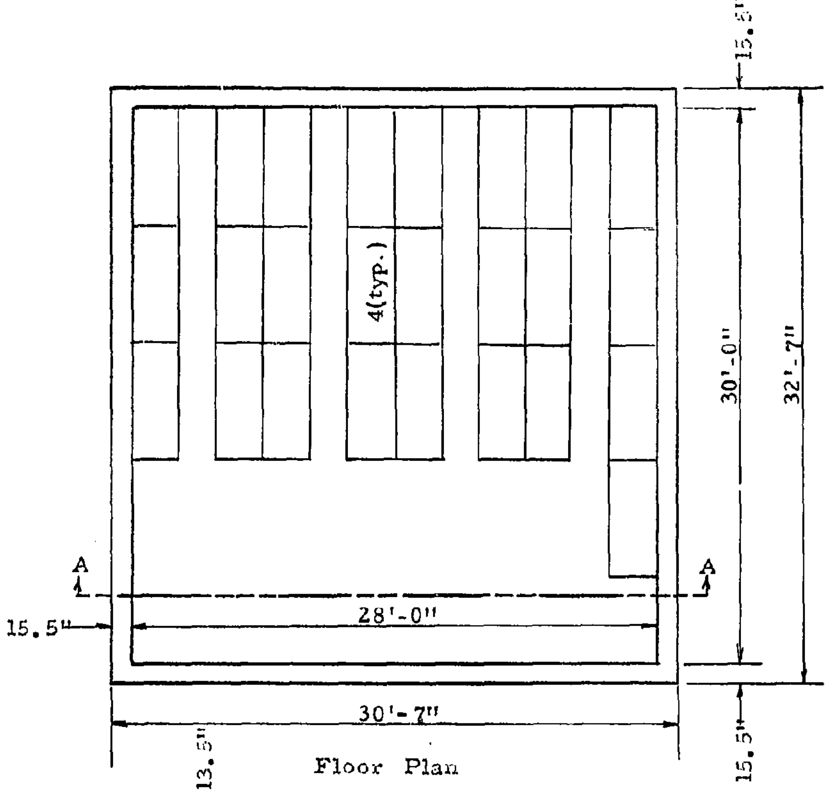
$\phi_{Sc} = \phi_{Lc}$ = 0.25 percent

D = 13.8 in.

For α = 1.0, L = 28.0 ft and q = 10 psi, these parameters are,

f_{dy} = 75,000 psi

f'_c = 2000 psi



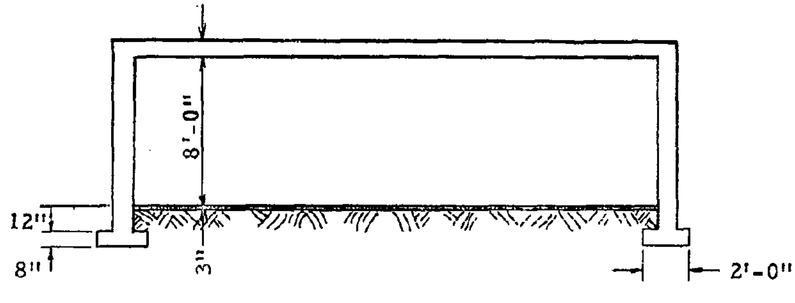


Figure 4-4

SHELTER LAYOUT TRIAL DESIGN 4.34C

$$\phi_{Sc} = \phi_{Lc} = 0.25 \text{ percent}$$

$$D = 13.1 \text{ in.}$$

For $\alpha = 0.933$, use $D = 13.5$ in.

Check for "pure" shear, 1680 lb/in. width < 7120 lb/in. width.

(b) Roof Slab Cost Factor

From Table 3-39, interpolating between $\alpha = 1.0$ and $\alpha = 0.9$, we obtain the unit cost of the roof slab.

$$C_t = 2.98 \text{ \$/sq ft}$$

(c) Eccentrically-Loaded Side and End Wall Design

Using guide lines expressed in Section 3.35, take $\phi_t = 0.50$ percent,

$$f'_{dc} = 2500 \text{ psi and } f_{dy} = 60{,}000 \text{ psi}$$

Required for use in Table 3-47

$$\frac{q}{f'_{dc}} = \frac{10}{2500} = 0.004$$

For $f_{dy} = 60{,}000$ psi and $f'_{dc} = 2500$ psi.

$$q_{d(wall)} = \frac{\phi'_t}{200} \quad \frac{f_{dy}}{f'_{dc}} = 0.12\, \phi'_t = 0.133\, \phi_t$$

From Section 4-22, $D \geq 6$ in. and $q_d \geq 0.067$. We next turn to Table 3-47, where the following design possibilities can be identified for $\frac{q}{f'_{dc}} = 0.004$ and $q_d \geq 0.067$.

(1) $q_d = 0.067$ $\quad \frac{d_{wall}}{L_{slab}} = 0.50$ (extrapolated)

(2) $q_d = 0.067$ $\quad \frac{d_{wall}}{L_{slab}} = 0.75$

The experience with Design Examples 4.34A and 4.34B suggests that the minimum-cost structural design for the side and end walls will consist of,

$$f_{dy} = 60,000 \text{ psi}$$

$$f'_{dc} = 2500 \text{ psi}$$

$$q_d = 0.067$$

$$\frac{d}{L} = 0.50$$

hence $d = 14.0$ in. and $D = 15.5$ in.

(d) Eccentrically-Loaded Side and End Wall Cost Factors

Concrete $C_c = \frac{15.5}{12} \times 1.00 = 1.29$ \$/sq ft from (3.35.33a)

Main Steel $C_s = \frac{0.50}{100} \times \frac{15.5}{12} \times 78.8 = 0.51$ \$/sq ft from (3.35.34c)

Temperature Steel $C_{st} = \frac{15.5}{12} \times 0.001 \times 78.8 = 0.10$ \$/sq ft from (3.35.35)

Forms $C_f = 1.00$ \$/sq ft from (3.35.36)

Summary

$$C_c = 1.29$$
$$C_s = 0.51$$
$$C_{st} = 0.10$$
$$C_f = \underline{1.00}$$
$$C_t = 2.90 \text{ \$/sq ft}$$

(e) Foundation Design for Eccentrically-Loaded Side and End Walls

Continuous footings are probably the most economical foundation system, because of the long span length.

Load acting on wall

$$P = 72\, q\, L$$

$$P = 72 \times 10 \times 28.0 = 20,200 \text{ lb/lineal ft of wall}$$

From Table 3-59 for $\phi = 15^{o}$, c = 2000 lb/sq ft and q = 10 psi, the ultimate bearing load of a footing with L = 2.0 ft is,

$$P = 24,000 \text{ lb/sq ft}$$

Actual load

$$\frac{P}{L} = \frac{20,200}{2.00} = 10,100 < 24,000$$

Required for use in Table 3-60.

$$\frac{P}{L} = 10,100 \text{ lb/ft/ft}$$

$$\frac{D_{wall}}{L_{footing}} = \frac{15.5}{2.0} = 7.75$$

Interpolating from Table 3-60 for f'_c = 2000 psi,
P/L = 10,100 lb and D/L = 7.75

$$\frac{d}{L} = 0.80 \qquad d = 2.00 \times 0.80 = 1.60 \text{ in.}$$

$$L = 2.00 \text{ ft}$$

A plain concrete footing with D = 8.00 in. as required by Section 4.22 is sufficient.

(f) Footing Cost Factors

Footing cost factors are determined in a manner identical to that used in Sample Design 4.34B

Summary	C_c =	$\frac{8}{12} \times 0.95 =$	0.63
	C_{st} =	$\frac{8}{12} \times \frac{0.10}{100} \times 78.8 =$	0.05
	C_f =		0.75
	C_t =		1.43 $/sq ft

(g) Floor Slab Design and Cost Factors

The structural parameters of the floor slab and the slab are the same as in Section (i) and (j) of Trial Design 4.34B.

$$C_t = 0.91 \text{ \$/sq ft}$$

(h) <u>Required Excavation</u>

1) Minimum cover h = 3.50 ft
2) Cover required for full burial h = 0.00 ft
3) Radiation burial requirement

$$d_e = 3.5 + (0.020 \times 10) = 3.70 \qquad \text{from (4.21.1b)}$$

For shielding, 1.0 ft of concrete $\approx$ 1.5 ft of earth

$$3.70 - \left(\frac{1.5D}{12}\right) = 3.70 - \left(\frac{1.5 \times 13.5}{12}\right) = 2.01 \text{ ft}$$

Use h = 3.50 ft

Total depth of excavation

$$z = 3.5 + 8.0 + \frac{13.5 + 3.0}{12} = 12.88 \text{ ft} \qquad \text{from (4.34.1)}$$

Following the criteria presented in Section 4.24,

$$\text{Volume} = \frac{12.88}{2}\left[(32.58 \times 30.58) + (58.34 \times 56.34)\right] = 27,600 \text{ cu ft}$$

from (4.21.1)

Cubicle gross volume

$$\text{Volume} = 32.58 \times 30.58 \left[8.0 + \left(\frac{13.5 + 3.0}{12}\right)\right] = 9340 \text{ cu ft}$$

from (4.34.2)

Trenching for footings

Use the same volume and cost factor shown in Trial Design 4.34B.

Length of trench = 2(32.58 + 28.00) = 121 ft
Volume of trench = 5.0 cu ft per lineal ft
C_t = 0.50 \$/lineal ft

(i) <u>Entrance Way</u>

From Table 4-4,

$$C_T = \$2750$$

(j) Total Cost

Roof Slab	2.98 x 32.58 x 30.58 =	2970
Side and End Walls	2.90 x 9.0 x 2(32.58 + 28.0)	3160
Floor Slab	0.91 x 30.00 x 28.00 =	756
Wall Footing	1.43 x 2.0 x 2(32.58 + 28.0) =	346
Footing Trench	0.50 x 121 =	61
Excavation	0.036 x 27,600 =	995
Back Fill	0.033 x (27,600 - 9340) =	603
Haul	0.026 x 9340 =	243
Entrance Way		2750
	Total	$11,884

Note: A raft foundation system would cost approximately $2500 as compared to the footing system which costs approximately $1550.

(k) Cubicle Net Floor Area

Net floor area = 28.0 x 30.0 = 840 sq ft

(l) Cubicle Net Volume

Net volume = 8.0 x 840 = 6720 cu ft

TRIAL DESIGN 4.34D

CONFIGURATION:

One story cubicle (see Figure 4-5)

STRUCTURAL SYSTEM:

Monolithic two-way slab-orthotropic-style A-quad(seeTable 4-3)

DESIGN PARAMETERS:

Same as Trial Design 4.34C. This example compares isotropic and orthotropic designs for the two-way reinforced slab.

(a) Roof Slab Design

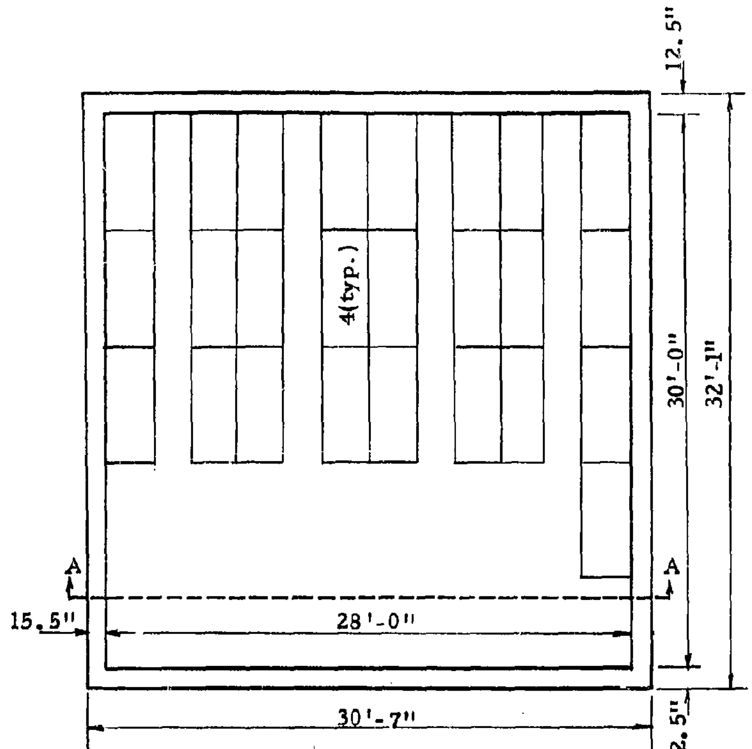
From Table 3-40, for L = 28.0 ft, α = 0.9 and q = 10 psi, the minimum-cost structural parameters for the overhead orthotropic ($\mu = \mu_c$) two-way slab with fixed-edge support and no web reinforcement are.

f_{dy} = 75,000 psi

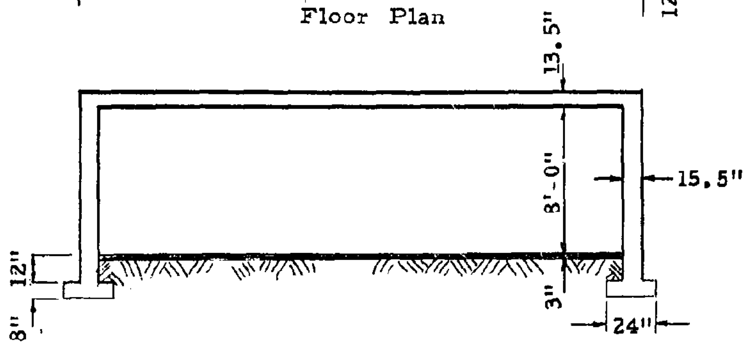
f'_c = 2000 psi

$\phi_{Sc} = \mu_e \phi_{Lc}$ = 0.25 percent

D = 13.8 in.



Floor Plan



Section A-A

Figure 4-5

SHELTER LAYOUT TRIAL DESIGN 4.34D

For $\alpha = 1.0$, $L = 28.0$ ft and $q = 10$ psi, these parameters are

$$f_{dy} = 75,000 \text{ psi}$$

$$f'_c = 2000 \text{ psi}$$

$$\phi_{Sc} = \mu_e \phi_{Lc} = 0.25 \text{ percent}$$

$$D = 13.1 \text{ in.}$$

Except for the introduction of μ_e, these are the same structural parameters as found for the isotropic slab of Trial Design 4-34C.

Use D = 13.5 in. The slab will be adequate in "pure" shear.

(b) Roof Slab Cost Factor

From Table 3-40, interpolating between $\alpha = 1.0$ and $\alpha = 0.9$, we obtain the unit cost of the roof slab.

$$C_t = 2.88 \text{ \$/sq ft}$$

(c) Eccentrically-Loaded Exterior Wall Design

(1) End Wall (long direction)

Using guide lines expressed in Section 3.35 take $\phi_t = 0.5$ percent (minimum), $f'_{dc} = 2500$ psi and $f_{dy} = 60,000$ psi

Required for use in Table 3-51

$$\frac{q}{f'_{dc}} = \frac{10}{2500} = 0.004$$

Also, for $f_{dy} = 60,000$ psi, $f'_{dc} = 2500$ psi and $\phi_t = \frac{\phi'_t}{0.9} = 0.50$ percent minimum, the minimum permissible value of q_d (wall) is 0.067. (see Trial Design 4-34C, Section C)

Interpolating from Table 3-51 for $q/f'_{dc} = 0.004$, $\alpha = 0.933$ and $q_d \geq 0.067$, the following design parameters offer logical choices for minimum-cost design.

$$q_d = 0.067 \qquad \frac{d_{wall}}{L_{slab}} = 0.40 \text{ (extrapolated)}$$

hence $d = 11.2$ in. and $D = 12.5$ in.

(Note that Table 3-51 and related tables should be extended to lower d/L ranges and be prepared for smaller d/L increments if they are to be used for design purposes).

(2) Side Wall (short direction)

Using guide lines expressed in Section 3.35, again take $\phi_t = 0.50$ percent (minimum), $f'_{dc} = 2500$ psi and $f_{dy} = 60,000$ psi.

Required for use in Table 3-55

$$\frac{q}{f'_{dc}} = \frac{10}{2500} = 0.004$$

Also, q_d(wall) = 0.067 minimum.

Interpolating from Table 3-55 for q/f'_{dc} = 0.004, α = 0.933 and $q_d \geqslant 0.067$, the probable minimum-cost parameters are,

$$q_d = 0.067 \qquad \frac{d_{wall}}{L_{slab}} = 0.50 \qquad \text{(extrapolated)}$$

hence d = 14.0 in. and D = 15.5 in.

(d) Eccentrically-Loaded Exterior Wall Cost Factors

Use the same method illustrated in Trial Design 4.34B (d)

(1) End wall

Summary

$$C_c = \frac{12.5}{12} \times 1.00 = \quad 1.04$$

$$C_s = \frac{0.50}{100} \times \frac{11.2}{12} \times 78.8 = \quad 0.37$$

$$C_{st} = \frac{0.10}{100} \times \frac{12.5}{12} \times 78.8 = \quad 0.08$$

$$C_f = \quad \underline{1.00}$$

$$C_t = \quad 2.49 \text{ \$/sq ft}$$

(2) Side wall

Use the same method illustrated in Trial Design 4.34B (d)

$$C_t = 2.85 \text{ \$/sq ft}$$

(g) Foundation Design and Cost Factors

Same as Trial Design 4.34C.

(h) Floor Slab Design and Cost Factors

Same as Trial Design 4.34C.

(i) Required Excavation

1) Minimum cover h = 3.50 ft
2) Cover required for full burial h = 0.00 ft
3) Radiation burial required

$$d_e = 3.5 + (0.020 \times 10) = 3.70 \qquad \text{from (4.21.1b)}$$

For shielding 1.0 ft of concrete $\approx$ 1.5 ft of earth

$$3.70 - \frac{1.5D}{12} = 3.70 - \frac{1.5 \times 13.5}{12} = 2.01 \text{ ft}$$

Use h = 3.50 ft

Total depth of excavation

$$z = 3.5 + 8.0 + \left(\frac{13.5 + 3.0}{12}\right) = 12.88 \text{ ft} \qquad \text{from (4.34.1)}$$

Following the criteria presented in Section 4.24

$$\text{Volume} = \frac{12.88}{2}\left[(32.08 \times 30.58) + (57.84 \times 56.34)\right] = 27,300 \text{ cu ft}$$

from (4.21.1)

Cubicle gross volume

$$\text{Volume} = 32.08 \times 30.58 \left[8.0 + \left(\frac{13.5 + 3.0}{12}\right)\right] = 9200 \text{ cu ft}$$

from (4.34.2)

Trenching for footing

Use the same volume and cost factor shown in Trial Design 4.34B.

Length of trench = 2(32.08 + 28.00) = 120 ft

(j) Entrance Way

From Table 4-4

C_T = \$2750

(k) Total Cost

Roof Slab	2.88 x 32.08 x 30.58 =	2820
Side Wall	2.85 x 9.0 x 30.00 x 2 =	1540
End Wall	2.49 x 9.0 x 30.58 x 2 =	1370
Floor Slab	0.91 x 30.00 x 28.00 =	765
Wall Footing	1.43 x 2.0 x 2(32.08 + 28.0) =	343
Footing Trench	0.50 x 2(32.08 + 28.0) =	60
Excavation	0.036 x 27,300 =	983
Backfill	0.033 x (27,300 - 9200) =	597
Haul	0.026 x 9200 =	239
Entrance Way		2750
	Total	\$11,467

(l) Cubicle Net Floor Area

Net floor area = 28.0 x 30.0 = 840 sq ft

(m) Cubicle Net Volume

Net volume = 8.0 x 840 = 6720 cu ft

TRIAL DESIGN 4.34E

CONFIGURATION:

One story cubicle (see Figure 4-6)

STRUCTURAL SYSTEM:

Flat slab(one-way modified) - style A - single with one interior partition. (see Table 4-3)

DESIGN PARAMETERS:

q = 10 psi equivalent pressure, including weight of slab and earth cover.

L = 7.0 ft

Design of all portions of the shelter is identical to Design Example 4.34A with the exception of the interior wall which is replaced by a column and drop panel system.

(a) Interior Column Design

$$P = 144\, q L^2 \text{ (approximate load)} \qquad (4.34.3)$$

$$P = 144 \times 10 \times (7.0)^2 = 70,500 \text{ lb}$$

$$P = 0.85\, A\, f'_{dc} + A_s f_{dy} \qquad (3.32.2)$$

Assume a 6 in. circular column with f'_{dc} = 2500 psi, f_{dy} = 44,000 psi. In order to ensure a reasonably-large area of reinforcing steel, use twice the minimum steel requirement, or ϕ_t = 1.0 percent.

$$P = (0.85 \times \pi \times 3^2 \times 2500) + (0.010 \times \pi \times 3^2 \times 44,000)$$

$$P = 72,400 \text{ lb.} \qquad \text{O.K.}$$

Use 6 in. circular column, f'_{dc} = 2500 psi, f_{dy} = 44,000 psi, ϕ_t = 1.00 percent. Concrete filled steel pipe or rolled structural steel shapes could be used instead of the concrete columns in many instances.

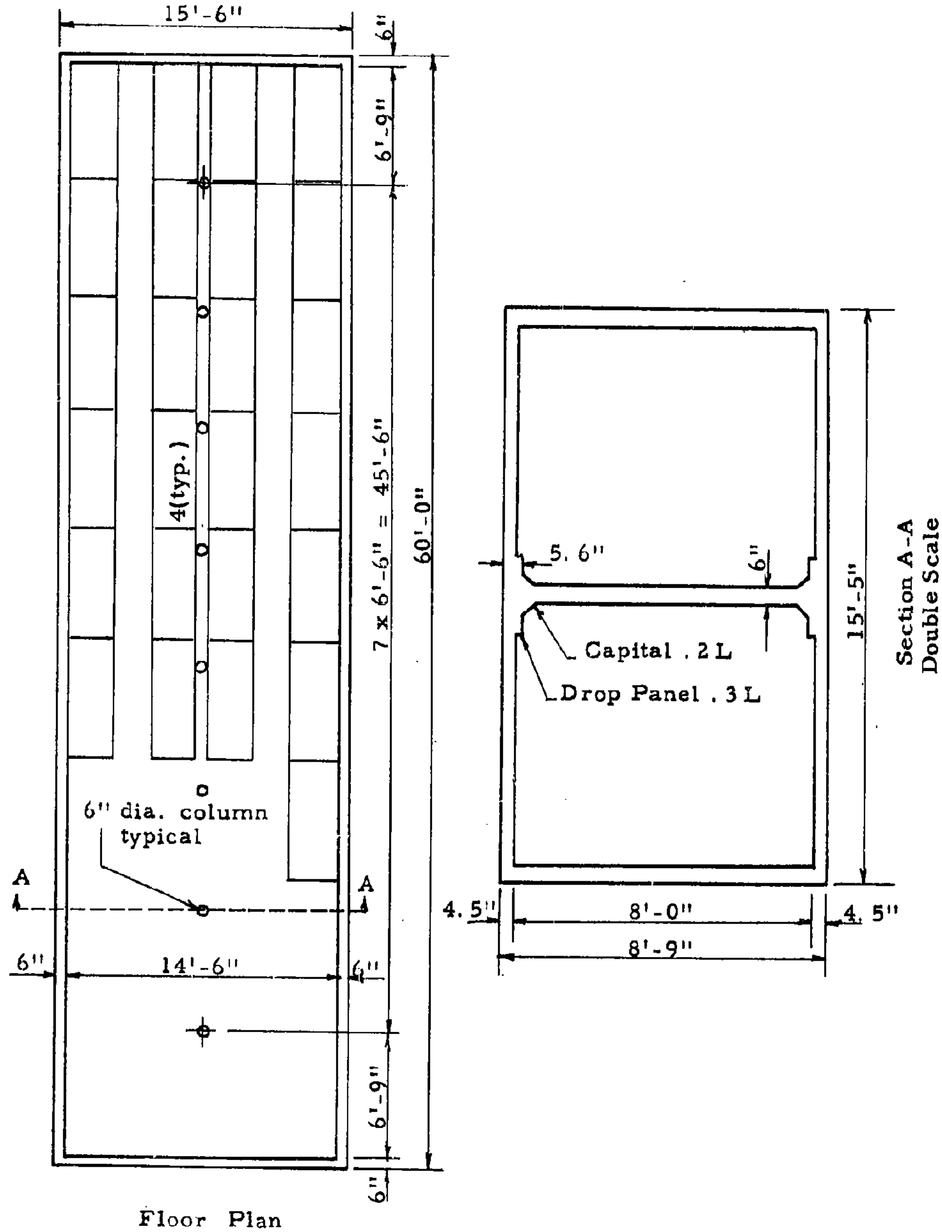


Figure 4-6

SHELTER LAYOUT TRIAL DESIGN 4.34E

(b) Interior Column Cost Factor

Concrete $$C_c = \left(\frac{A}{144}\right) X_c \qquad (3.32.4a)$$

$$C_c = \frac{28.3}{144} \times 1.00 = 0.20 \text{ \$/ft}$$

Main Steel $$C_s = \frac{A}{144}\left(\frac{\phi_t}{100}\right) X_s \qquad (3.32.5a)$$

$$C_s = \frac{28.3}{144} \times 0.010 \times 78.8 = 0.16 \text{ \$/ft}$$

Tie Steel (For convenience in concrete members which require tie steel but no temperature steel, the cost equation for temperature steel is used to evaluate the cost factor for tie steel.)

$$C_{st} = \frac{A}{144}\left(\frac{\phi_{te}}{100}\right) X_s \qquad (3.32.6a)$$

$$C_{st} = \frac{28.3}{144} \times 0.001 \times 78.8 = 0.02 \text{ \$/ft}$$

Forms $$C_f = P_r X_f \qquad (3.32.7a)$$

$$C_f = 3.14 \times \frac{6.00}{12} \times 1.10 = 1.73 \text{ \$/sq ft}$$

Summary

$$C_c = 0.20$$
$$C_s = 0.16$$
$$C_{st} = 0.02$$
$$C_f = \underline{1.73}$$
$$C_t = 2.11 \text{ \$/ft of column}$$

See Figure 4-6 for number of columns required.

(c) Drop Panel Design

The flexure coefficient k_f and the diagonal tension coefficient k_{sc} for the flat slab arrangement chosen should at least equal the flexure and diagonal tension coefficient for the one-way slab, so as to preclude a premature failure in the region surrounding the interior column and drop panel.

Assume:

Drop Panel $\quad P_p = 0.3L \quad d_p = 1.25d$

Capital $\quad D_c = 0.2L$

Check for flexure

From Tables 3-57 and 3-38

k_f (flat slab) = 0.00162

k_f (one-way slab) = 0.00100 < 0.00162 (O.K. for flexure)

k_{sc} (slab-drop panel interface) = 3.421

k_{sc} (drop panel-capital interface) = 2.360 (modified)

k_{sc} (one-way slab) = 1.765 < 2.360 < 3.421 ∴ O.K. for diagonal tension

It must be noted that the shear compression resistance, q_{sc} (Equation 3.34.34), of the drop panel is related to that of the slab by the ratio $(d_p/d)^2$. In this example the equivalent k_{sc} for the drop panel-capital interface can be related to that of the one-way slab by

$$(1.25)^2 \, k_{sc} = 1.56 \times 1.513 = 2.360$$

(d) Two-Way Slab Strip Additional Cost Factors

To provide for flat slab action in a one-way slab, a slab strip D_c wide running parallel to the column line is provided with two-way reinforcement.

$$C'_s = 2\,C_s$$

where

C'_s = the cost factor for two-way reinforcement in the slab strip running between drop panels, (\$/sq ft)

C_s = the cost factor for the reinforcement in the one-way slab, (\$/sq ft)

From Design Problem 4.34A, Section (a), we obtain the structural parameters for the one-way reinforced slab. From these, by applying Equation 3.33.30b, we can obtain the cost factor C'_s.

$$C'_s = \frac{2X_s \phi_c d}{1200}\left[1.33 + \frac{0.278}{L}\,\frac{f_{dy}}{f'_c}\right] = \left(\frac{2 \times 85.8 \times 0.40 \times 4.0}{1200}\right) \times \left[1.33 + \frac{0.278 \times 75,000}{6.0 \times 3600}\right] = 0.526 \ \$/\text{sq ft}$$

Additional cost factor for two-way reinforcement

$$\Delta C_t = \frac{C'_s}{2} = \frac{0.526}{2} = 0.26 \ \$/\text{sq ft}$$

(e) Drop Panel Additional Cost Factors

Concrete $$C_c = \left(\frac{D_p}{D} - 1\right) C_c \text{ (slab)} \qquad \text{(see 3.36.13)}$$

From Design Problem 4-34A and Equation 3.33.30a.

$$C_c \text{ (slab)} = \frac{4.5}{12} \times 1.18 = 0.44 \ \$/\text{sq ft}$$

Therefore, the additional cost factor for concrete in the drop panel is,

$$C_c = \left(\frac{5.50}{4.50} - 1\right) \times 0.44 = 0.10 \ \$/\text{sq ft}$$

Main Steel $$C_s = \left(\frac{d_p}{d} - 1\right) C'_s \text{ (slab)} \qquad \text{(see 3.36.13)}$$

$$C_s = \left(\frac{4.30}{3.44} - 1\right) \times 0.526 = 0.13 \ \$/\text{sq ft}$$

(f) Capital Cost Factor

$$C_T = 0.00052\,(X_c + 0.01\ X_s)\,L^3 + 0.0332 X_f L^2 \qquad (3.36.14a)$$

$$C_T = 0.00052\left[1.18 + (0.01 \times 78.8)\right] 6^3 + 0.0332 \times 1.75 \times 6^2$$

$$C_T = 0.22 + 2.09 = 2.31 \ \$/\text{capital}$$

Summary

$$C_c = 0.10$$

$$C_s = \underline{0.13}$$

$$C_t = 0.23 \ \$/\text{sq ft}$$

(g) Total Cost of Flat Slab Drop Panel System

Columns	2.11 x 8.0 x 8 =	135
Capitals	2.31 x 8 x 2 =	37
Drop Panels	0.23 x 0.094 x 49 x 8 x 2 =	17
Two Slab Strip	0.26 x 0.3 x 7.0 x 49.0 x 2 =	54
Additional Form	0.07 x 15.5 x 60.0 x 2 =	130
	Total	$373

This cost compares favorably with the $477 cost for the interior partition shown in Trial Design 4.34A.

Total cost of shelter- $8517

(h) Cubicle Net Floor Area

Net area = (14.5 x 59.0) - (8 x 0.785 x 0.25) = 855 - 1.6 = 853 sq ft.

(i) Cubicle Net Volume

Net volume, neglecting effect of capitals and drop panels, is approximately 6800 cu ft.

TRIAL DESIGN 4.34F

CONFIGURATION:

One story cubicle (see Figure 4-7)

STRUCTURAL SYSTEM:

Reinforced concrete framing system-style A-single with one interior support. (see Table 4-3)

DESIGN PARAMETERS:

q = 10 psi equivalent pressure including weight of slab and earth cover

L = 7.0 ft

(a) Roof and Raft Foundation Design and Cost Factors

Same as Trial Design 4.34A.

(b) Exterior Beam Design

Table 3-15 supplies minimum-cost structural parameters for one-way reinforced, overhead slabs with fixed edge support, while the material relationships shown in this table may be of some assistance in estimating minimum-cost parameters for one-way reinforced beams, it must be recognized that definitive solutions to the problem of minimizing costs in one-way beams would involve a separate cost study. Such a study has not been undertaken in the current program. The ratio of beam width, b, to beam effective depth, d, would enter into such a relationship. It is conventional practice, subject to restrictions imposed by layout requirements, to design reinforced concrete beams with depths which are 1.5 to 2.5 times their width. These relationships, which have not received full verification by cost minimization studies, could conceivably be altered when concepts of ultimate strength and

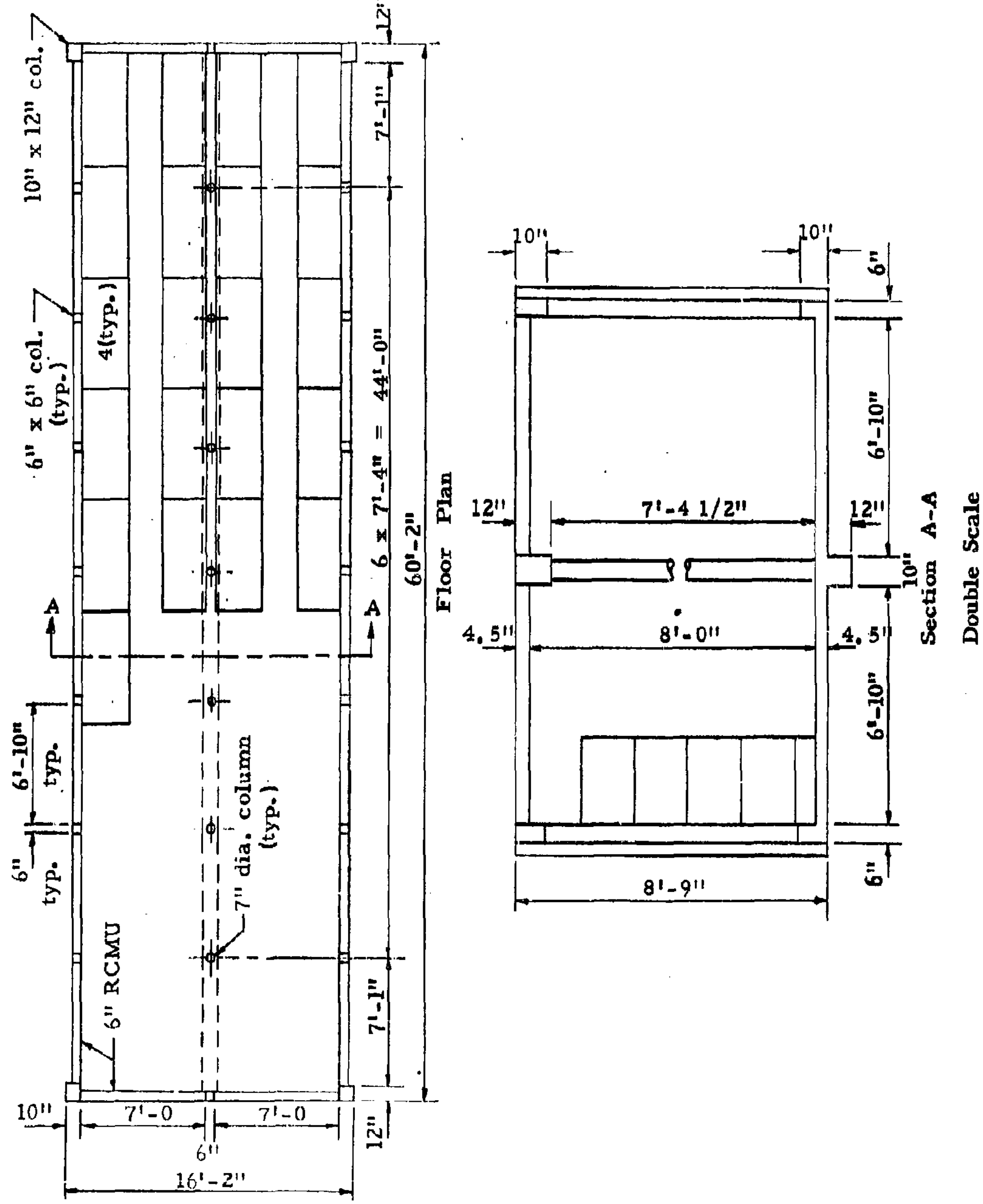


Figure 4-7

SHELTER LAYOUT TRIAL DESIGN 4.34F

progressive plastic yielding at beam sections are introduced. For the design example, unlike conventional construction, it is advantageous from the standpoint of interior space layout to use a deep beam whose width is restricted to approximately that of the exterior filler wall. The restraint afforded such a beam will be such that local buckling should not occur, nor is "pure" shear anticipated to be a problem. However, it is almost certain that these relative beam dimensions would not be associated with the true "minimum cost" beam.

Reference 9 suggests 7.0 ft as the shortest practical span length and, since the shorter span lengths are generally associated with maximum beam economy, columns will be located to provide spans of approximately 7.0 ft. (see Figure 4-7) Assuming a 6 in. column dimension along the longitudinal beam axis, the clear-span length of beam will be 6'-10" for 7 columns spaced at 7'-4" on centers. Full beam-column moment transfer is postulated.

Initially, assume width of 8 in. for the beam. From Figure 4-7, for L = 6.83 ft (clear span) and q_{slab} = 10 psi, the unit load per square inch of beam can readily be computed.

$$q_{beam} = \frac{10}{8} \left[8 + \left(\frac{7.0 \times 12}{2} \right) \right] = 62.5 \text{ psi}$$

From Table 3-15, for L = 7.0 ft and q = 50 psi, the pertinent structural parameters associated with minimum in-place cost for fixed-end overhead slabs are f_{dy} = 75,000 psi, ϕ_v = 0, and f'_c = 6000 psi. These should supply some guidance as to minimum-cost structural parameters for the beam. However, we probably should also investigate a case with minimum web reinforcement (ϕ_v = 0.50 percent).

Next, assume a width of 6 in. for the beam. The corresponding beam unit loading is,

$$q_{beam} = \frac{10}{6} \left[6 + \left(\frac{7.0 \times 12}{2} \right) \right] = 80 \text{ psi}$$

From Table 3-15, for L = 7.0 ft and q = 75 psi, the pertinent structural parameters are f_{dy} = 75,000 psi, ϕ_v = 0, and f'_c = 5900 psi. Again, in considering the minimum-cost beam, the ϕ_v = 0.50 percent case should also be investigated.

Turning to Table 3-13, with θ' = 0.25 for all cases, we can identify four design situations for study.

(1) $L = 6.83$ ft, $q = 62.5$ psi, $f_{dy} = 75,000$ psi, $f'_c = 6000$ psi, $b = 8.0$ in.

$$\phi_v = 0$$

$$\phi_c = 0.656 \text{ percent}$$

$$d = 7.65 \text{ in.}$$

$$D = \frac{d}{0.9} = 8.5 \text{ in.}$$

(2) $L = 6.83$ ft, $q = 62.5$ psi, $f_{dy} = 75,000$ psi, $f'_c = 2000$ psi, $b = 8.0$ in.

$$\phi_v = 0.50 \text{ percent}$$

$$\phi_c = 0.67 \text{ percent}$$

$$d = 7.65 \text{ in.}$$

$$D = 8.5 \text{ in.}$$

Case (2) is almost certainly more expensive than Case (1), so will be dropped from further consideration.

(3) $L = 6.83$ ft, $q = 80$ psi, $f_{dy} = 75,000$ psi, $f'_c = 6000$ psi, $b = 6$ in.

$$\phi_v = 0$$

$$\phi_c = 0.656 \text{ percent}$$

$$d = 8.75 \text{ in.}$$

$D = 9.75$ in. - say, 10.0 in.

(4) $L = 6.83$ ft, $q = 80$ psi, $f_{dy} = 75,000$ psi, $f'_c = 2000$ psi, $b = 6$ in.

$$\phi_v = 0.5 \text{ percent}$$

$$\phi_c = 0.67 \text{ percent}$$

$$d = 8.75 \text{ in.}$$

$D = 9.75$ in. - say, 10.0 in.

Again, Case (4) is almost certainly more expensive than for Case 3.

(c) Exterior Beam Cost Factors

Cost comparisons will be made only for design cases 1 and 3. Since the roof slab and the beam will be cast monolithically, the beam can be costed as a localized increase in slab depth (modified T-beam analysis) or the beam can be costed separately and the slab clear-span reduced by one-half the beam width. However, in recognition of the increased costs of fabricating the

beam-slab connection, we will compute the beam cost as a discrete item and will also assume that the slab unit costs corresponding to L = 7.0 ft and q_{slab} = 10 psi are still applicable over the reduced effective roof area.

(1) Beam Design No. 1

Concrete $$C_C = \frac{X_c\ DLb}{144} \qquad (3.33.22)$$

$$C_C = \frac{1.37 \times 8.5 \times 6.83 \times 8.0}{144} = \$4.42$$

Main reinforcement $$C_S = \frac{X_s\ dLb\ \phi_c}{14,400}\left[1.33 + \frac{0.278}{L}\ \frac{f_{dy}}{f'_c}\right]$$

$$C_S = \left[\frac{85.8 \times 7.65 \times 6.83 \times 8.0 \times 0.656}{144 \times 100}\right]\left[1.33 + \left(\frac{0.278 \times 75,000}{6.83 \times 6000}\right)\right]$$
$$= \$3.01$$

Web Steel None required.

Temperature Steel None required.

Form Work $$C_F = k'_f\ L\ \frac{(b + D)}{6} \qquad (3.33.29b)$$

$$C_f = \left[0.88 + (0.012 \times 8.5)\right] \times 6.83 \times \left(\frac{8.0 + 8.5}{6}\right) = \$18.44$$

Summary
$$C_C = 4.42$$
$$C_S = 3.01$$
$$C_F = \underline{18.44}$$
$$C_T = \$25.87$$
$$C_t = \frac{C_T}{L} = \frac{25.87}{6.83} = 3.79\ \$/\text{ft}$$

(2) Beam Design No. 3.

Concrete $$C_C = \frac{1.37 \times 10.0 \times 6.83 \times 6.0}{144} = \$3.90$$

Main Reinforcement
$$C_S = \left[\frac{85.8 \times 8.75 \times 6.83 \times 6.0 \times 0.656}{144 \times 100}\right]\left[1.33 + \left(\frac{0.278 \times 75,000}{6.83 \times 6000}\right)\right]$$
$$= \$2.58$$

Web Steel None required.

Temperature Steel None required.

Form Work $$C_F = \left[0.88 + (0.012 \times 10.0)\right] \times 6.83 \times \left(\frac{6.0 + 10.0}{6}\right)$$
$$= \$18.20$$

Summary

$$C_C = 3.90$$

$$C_S = 2.58$$

$$C_F = \underline{18.20}$$

$$C_T = \$24.68$$

$$C_t = \frac{24.68}{6.83} = 3.62 \ \$/\text{ft}$$

Use Trial Design No. 3., with b = 6 in.

(d) Interior Beam Design

From the standpoint of interior space usage, it would be desirable to have a shallow interior beam. As an initial trial, assume b = 10 in. For this beam width and 10 psi roof loading, the unit loading on the beam (see Figure 4-7) is,

$$q_{beam} = \frac{10}{10}\left[10 + (6.83 \times 12)\right] = 92 \text{ psi}$$

Table 3-15, for L = 7.0 ft and q = 100 psi, indicates that f_{dy} = 75,000 psi, ϕ_v = 0 and f'_c = 6000 psi are associated with the least-cost slab. Entering Table 3-13 with these values, and assuming θ' = 0.25, a possible structural design for the interior beam is as follows,

$$f_{dy} = 75,000 \text{ psi}$$

$$f'_c = 6000 \text{ psi}$$

$$\phi_v = 0$$

$$\phi_c = 0.656 \text{ percent}$$

Conservatively assuming L = (6.83 + 0.50) = 7.33 ft we obtain $d \simeq 10.5$ in. and $D = \frac{d}{0.9} = 11.7$ in.

Use d = 10.5 in. and D = 12.0 in.

Check for "pure" shear. Maximum shear at fixed support is

$$V_{max} = \frac{10 \times 7.46 \times 12 \times 7.5 \times 12}{2} = 40,300 \text{ lb}$$

$$V_{allowable} = 0.22 \times 6000 \times 10.0 \times 10.5 = 139,000 \text{ lb} \qquad \text{O.K.}$$

(e) Interior Beam Cost Factor

Concrete $$C_C = \frac{X_c D L b}{144} \qquad (3.33.22)$$

$$C_C = \frac{1.37 \times 12.0 \times 7.33 \times 10.0}{144} = \$8.37$$

Main Reinforcement

$$C_S = \frac{X_s\, d\, L\, b\, \phi_c}{14,400}\left[1.33 + \frac{0.278}{L}\ \frac{f_{dy}}{f'_c}\right]$$

$$C_S = \left[\frac{85.8 \times 10.50 \times 7.33 \times 10.0 \times 0.656}{14,400}\right]\left[1.33 + \left(\frac{0.278 \times 75,000}{7.33 \times 6000}\right)\right]$$

$$= \$5.43$$

Web Steel None required.

Temperature Reinforcement None required.

Form Work $C_F = k'_f\, L\, \frac{(b + D)}{6}$ (3.33.29b)

$$C_F = \left[0.88 + (0.012 \times 12.0)\right] \times 7.33 \times \left(\frac{10.0 + 12.0}{6}\right) = \$27.50$$

Summary

$$C_C = 8.37$$

$$C_S = 5.43$$

$$C_F = \underline{27.50}$$

$$C_T = \$41.30$$

$$C_t = \frac{C_T}{L} = \frac{41.30}{7.33} = 5.63\ \$/\text{ft}$$

(f) <u>Design of Axially-Loaded Interior Column</u>

The design of the interior columns is controlled by the load on the first interior column, which supports the end reaction of a 7.33 ft clear-span length of interior beam.(see Figure 4-7) Including the direct load on the column from the roof slab, the total interior column load is,

$$P_{col} = (7.46 \times 7.50 \times 144 \times 10)$$
$$= 80,600 \text{ lb}$$

$$P_{allow} = 0.85\, A\, f'_{dc} + A_s\, f_{dy} \quad (3.32.2)$$

Assume a 7 in. diameter circular column with $f'_{dc} = 2500$ psi ($f'_c = 2000$ psi), $\phi_t = 1.00$ percent (in excess of 0.50 percent minimum because of small cross-sectional area of column), and $f_{dy} = 44,000$ psi

$$P_{allow} = (0.85 \times \pi \times 3.5^2 \times 2500) + (0.010 \times \pi \times 3.5^2 \times 44,000)$$
$$= 98,700 \text{ lb}$$

Use 7 in. diameter circular column, f'_{dc} = 2500 psi, f_{dy} = 44,000 psi, ϕ_t = 1.00 percent.

(g) Cost Factor For Axially-Loaded Interior Column

Concrete $$C_c = \left(\frac{A}{144}\right) X_c \qquad (3.32.4a)$$

$$C_c = \frac{0.785 \times 7^2}{144} \times 1.00 = 0.27 \text{ \$/ft}$$

Main Steel $$C_s = \frac{A}{144}\left(\frac{\phi_t}{100}\right) X_s \qquad (3.32.5a)$$

$$C_s = \frac{0.785 \times 7^2}{144} \times 0.010 \times 78.8 = 0.21 \text{ \$/ft}$$

Tie Steel $$C_{st} = \frac{A}{144}\left(\frac{\phi_{te}}{100}\right) X_s \qquad (3.32.6a)$$

$$C_{st} = \frac{0.785 \times 7^2}{144} \times 0.001 \times 78.8 = 0.02 \text{ \$/ft}$$

Forms $$C_f = P_r X_f \qquad (3.32.7a)$$

$$C_f = \frac{7\pi}{12} \times 1.10 = 2.01 \text{ \$/ft}$$

Summary

$$C_c = 0.27$$
$$C_s = 0.21$$
$$C_{st} = 0.02$$
$$C_f = \underline{2.01}$$
$$C_t = 2.51 \text{ \$/ft}$$

(h) Eccentrically-Loaded Corner Column Design

The eccentricity of the exterior beam reaction on the corner columns must be considered in the design of these members. From Section (b) of this Design Example, we see that the loading on the exterior beam is 80 psi, over a beam width of 6 in. and a clear-span length of 6.83 ft for assumed column dimensions of 6 in. parallel to the longitudinal axis of the beam. We will now determine the actual required dimensions for the corner column. In so doing, we will check the earlier assumption of 6.83 ft for the clear-span length of the exterior beam.

The exterior beam loading is applied to the corner column over the 6 in. width of exterior beam. Hence, assuming that the column width perpendicular to the exterior beam is 10 in., the equivalent loading for column design by Tables 3-41 to 3-44 is,

$$q_{column} = \frac{6}{10} \quad q_{beam} = \frac{6 \times 80}{10} = 48 \text{ psi}$$

Following the guide lines presented in Section 3.35 and utilized in other design examples, take minimum ϕ_t = 0.50 percent, f'_{dc} = 2500 psi, and f_{dy} = 60, 000 psi.

Required for use in Table 3-43

$$\frac{q}{f'_{dc}} = \frac{48}{2500} = 0.0192$$

$$\text{minimum } q_d = \frac{1}{2}\left(\frac{\phi'_t}{100}\right)\frac{f_{dy}}{f'_{dc}} = \frac{1}{2} \times \frac{0.0050}{0.9} \times \frac{75,000}{2500} = 0.083$$

From Table 3-43 for q/f'_{dc} = 0.0192 and q_d = 0.083, we read $\frac{d_{col}}{L_{slab}}$ = 1.5. Assuming L = 6.83 ft, as before, we thus obtain

$$d_{col} = 1.5 \times 6.83 = 10.2 \text{ in.}$$

$$D = \frac{d}{0.9} = \frac{10.2}{0.9} = 11.3 \text{ in.}$$

Use a 10 in. x 12 in. corner column. Supply 0.50 percent steel in each major bending plane, or total ϕ_t = 1.0 percent.

(i) Eccentrically-Loaded Corner Column Cost Factors

Concrete $$C_c = \left(\frac{b\,D}{144}\right) X_c \qquad (3.35.33b)$$

$$C_c = \frac{10.0 \times 12.0}{144} \times 1.00 = 0.83 \text{ \$/ft}$$

Main Steel $$C_s = \frac{b\,d}{144}\left(\frac{\phi'_t}{100}\right) X_s \qquad \text{from (3.35.34b)}$$

$$C_s = \frac{10.0 \times 12.0}{144} \times \frac{0.010}{0.9} \times 78.8 = 0.73 \text{ \$/ft}$$

Tie Steel

$$C_{st} = \frac{b\,D}{144} \left(\frac{\phi_{te}}{100} \right) X_s \qquad (3.35.35b)$$

$$C_{st} = \frac{10.0 \times 12.0}{144} \times 0.001 \times 78.8 = 0.07 \ \$/ft$$

Forms

$$C_f = \left(\frac{b + D}{6} \right) X_f \qquad (3.35.36b)$$

$$C_f = \left(\frac{10 + 12}{6} \right) \times 1.00 = 3.67 \ \$/ft$$

Summary

$$\begin{aligned} C_c &= 0.83 \\ C_s &= 0.73 \\ C_{st} &= 0.07 \\ C_f &= \underline{3.67} \\ C_t &= 5.30 \ \$/ft \end{aligned}$$

(j) Design of Exterior Wall Columns (Non-Corner)

These columns are loaded by the end reactions of the exterior beams, plus an additional loading due to the strip of roof slab which bears directly on the column. The first loading component has an axial resultant on the column, while the second loading component has a small eccentricity. However, since the eccentric loading component is proportionately minor, it can be assumed that the minimum requirement for column steel (ϕ_t = 0.50 percent) will be adequate. Thus, the column is designed for an axial loading P_{column} where,

$$P_{column} = \left(\frac{2 \times 6.0 \times 6.83 \times 12 \times 80}{2} \right) + \left(\frac{7.0 \times 12 \times 6 \times 10}{2} \right)$$

$$= 39,300 + 2520 = 41,820 \ \text{lb}$$

A column width of 6 in. is desirable from the standpoint of interior layout. Neglecting the portion of the load carried by the column steel, the required gross depth of column for f'_{dc} = 2500 psi concrete would be

$$D = \frac{41,820}{6.0 \times 0.85 \times 2500} = 3.28 \ \text{in.}$$

Use D = 6 in. with ϕ_t = 0.50 percent in each principal bending plane.

A column width of 6 in. was assumed in the analysis of the exterior beam, and the adequacy of this interior beam is now verified.

(k) Cost Factor For Exterior Wall Columns(Non-Corner)

Concrete
$$C_c = \left(\frac{b\,D}{144}\right) X_c \qquad (3.35.33b)$$

$$C_c = \frac{6.0 \times 6.0}{144} \times 1.00 = 0.25 \ \$/ft$$

Main Steel
$$C_s = \left(\frac{\phi_t\, b\, D}{14,400}\right) X_s$$

$$C_s = \frac{1.00 \times 6.0 \times 6.0}{14,400} \times 78.8 = 0.20 \ \$/ft$$

Tie Steel
$$C_{st} = \frac{b\,D}{144}\left(\frac{\phi_{te}}{100}\right) X_s \qquad (3.35.35b)$$

$$C_{st} = \frac{6.0 \times 6.0}{144} \times 0.001 \times 78.8 = 0.02 \ \$/ft$$

Forms
$$C_f = \left(\frac{b + D}{6}\right) X_f \qquad (3.35.36b)$$

$$C_f = \left(\frac{6 + 6}{6}\right) \times 1.00 = 2.00 \ \$/ft$$

Summary

$C_c = 0.25$

$C_s = 0.20$

$C_{st} = 0.02$

$C_f = \underline{2.00}$

$C_t = 2.47$ \$/ft

(1) Concrete Masonry Wall Design

Using the arching theory developed in Section 3.5, the masonry unit wall is designed as a bending member to resist a component of load equal to one-half that acting on the roof slab. The span length is that taken in the short direction.

Assume D = 6.00 in. (6 in. RCMU)

f'_{cm} = 1000 psi, E = 1,000,000 psi

Maximum design span length for the masonry wall occurs at end walls, where the clear-span length between the corner column and the exterior(non-corner) column is 7.0 ft.

Following the procedure outlined in Section 3.5

$$\frac{12L}{D} = \frac{12 \times 7.00}{6.00} = 14.0$$

$$e'_{cm} = \frac{f'_{cm}}{E} = \frac{1000}{1,000,000} = 0.001$$

$$R = \frac{e'_{cm}}{4}\left(\frac{12L}{D}\right)^2 = \frac{0.001}{4} \times (14.0)^2 = 0.049$$

From Table 3-66 for R = 0.049 and 12L/D = 14.0

$$\frac{q}{f'_{cm}} = 0.0075 \qquad q = 1000 \times 0.0075 = 7.5 \text{ psi}$$

The value of q acting on the wall is assumed equal to one-half of that acting on the roof slab.

$$\frac{q}{2} = \frac{10}{2} = 5 < 7.5 \therefore \text{ O.K.}$$

Use 6 in. RCMU for all exterior walls.

(m) Concrete Masonry Wall Cost Factor

6 in. RCMU, C_t = 1.10 \$/sq ft.

(n) Foundation Design and Cost Factors

Same as Trial Design 4.34A.

(o) Required Excavation

Required depth z is the same as that shown in Trial Design 4.34A.

$$z = 12.25 \text{ ft}$$

Following the criteria presented in Section 4.24

$$\text{Volume} = \frac{12.25}{2}\left[(60.17 \times 16.17) + (84.67 \times 40.67)\right] = 27,000 \text{ cu ft}$$

Cubicle gross volume

Approximately 60.17 x 15.5 x 8.75 = 8150 cu ft

(p) Entrance Way

From Table 4-4

$$C_t = \$2750$$

(q) Total Cost

Roof Slab	1.66 x 13.67 x 59.17 =	1343
Ground Slab	1.33 x 13.67 x 59.17 =	1075
Exterior Beam	3.62 x [(4 x 6.83 x 8) + (4 x 7.00 x 2)] =	995
Interior Beam	5.63 x 2 x 59.17 =	666
Corner Columns	5.30 x 4 x 8.75 =	186
Exterior (Non-Corner) Columns	2.47 x 8.75 x 16 =	346
Interior Columns	2.51 x 7.38 x 12 =	222
End Walls	1.10 x 2 x 2 x 7.0 x 7.08 =	218
Side Walls	1.10 x 2 x 8.0 x 6.83 x 7.08 =	850
Excavation	0.036 x 27,000 =	972
Backfill	0.033 x (27,000 - 8150) =	622
Haul	0.026 x 8150 =	212
Entranceway		2750
	Total	$10,457

(r) Cubicle Net Floor Area

Net floor area = $(14.5 \times 59.17) - (7 \times 0.785 \times 0.667^2) = 859 - 2 = 857$ sq ft

(s) Cubicle Net Volume

Net volume = $(857 \times 8.0) - \left(\frac{7.5 \times 10.0}{144} \times 59.17\right) = 6850 - 31 = 6819$ cu ft

TRIAL DESIGN 4.34G

CONFIGURATION:

One story cubicle (see Figure 4-8)

STRUCTURAL SYSTEM:

Structural steel framing system-style A-single with one interior partition.(see Table 4-3)

DESIGN PARAMETERS:

q = 10 psi equivalent pressure including weight of slab and earth cover

L = 7.0 ft

(a) Roof and Ground Slab Design and Cost Factors

Same as Trial Design 4.34 A

(b) Interior Beam Design

Assume beam width = 4.00 in., L = 6.50 ft (from Design Example 4.34 E)

From Section 3.24 assume $L \leq L_{fv}$ (shear governs)

$$\frac{q_v B L}{f_{dy}} = \frac{10 \times 7.33 \times 6.50}{44,000} = 0.0108$$

Try 10 Jr 9

From Table 3-3

$$\frac{q_v B L}{f_{dy}} = 0.01242 > 0.0108 \therefore \text{O.K.}$$

Assuming the use of an interior column with D > 6.00 in., the clear span L between columns become 6.50 - 0.50 = 6.00 > 5.55. From Table 3-3 for a 10 Jr 9 moment governs for $L \geqq L_{fv}$.

$$\frac{q_b B L^2}{f_{dy}} = \frac{10 \times 7.33 \times (6.00)^2}{44,000} = \frac{2640}{44,000} = 0.0600$$

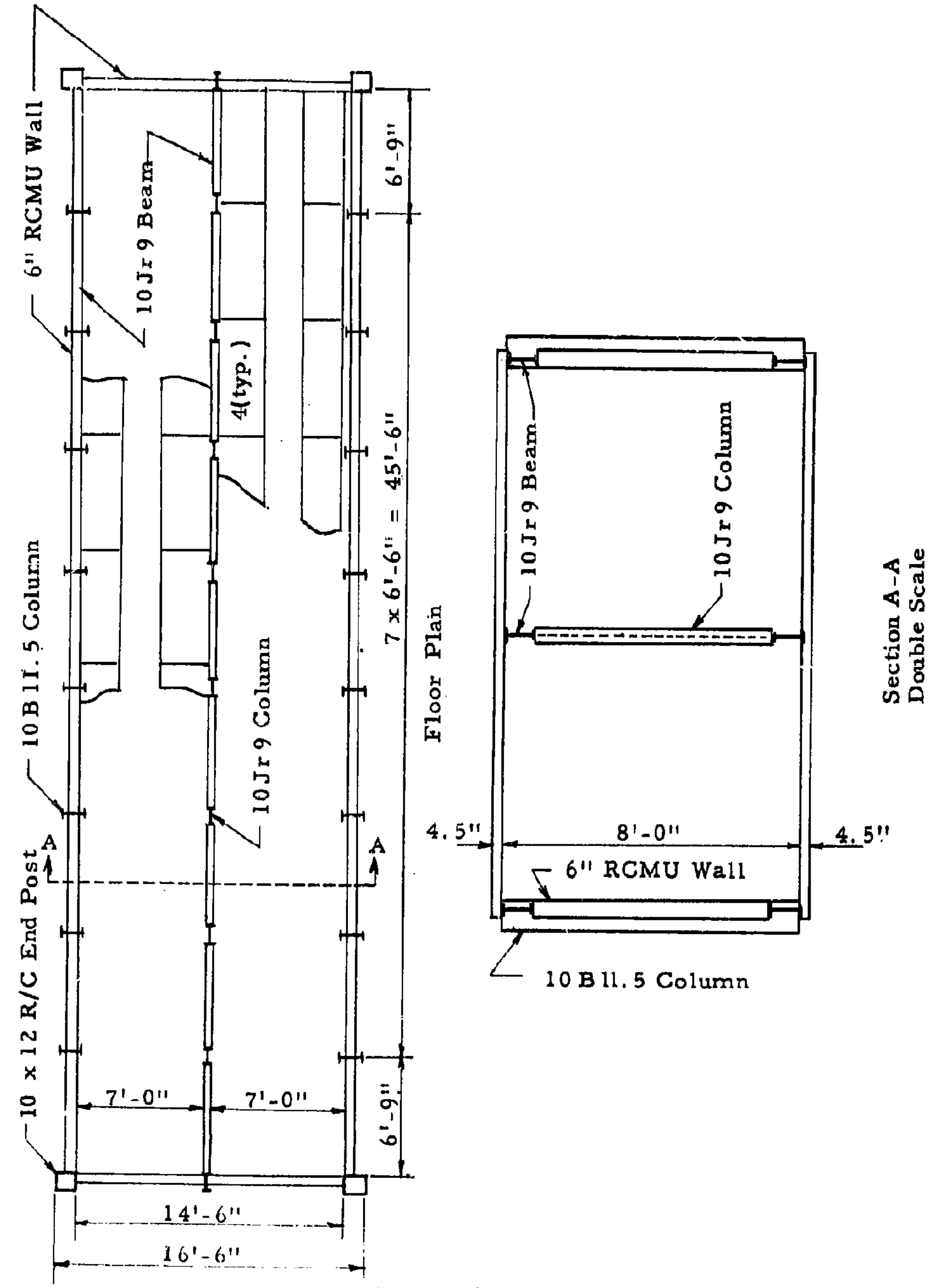


Figure 4-8

SHELTER LAYOUT TRIAL DESIGN 4.34G

From Table 3-3 for $L > L_{fv}$, 10 Jr 9

$$\frac{q_f B L^2}{f_{dy}} = 0.06893$$

$.06893 > .0600$ required $\therefore$ O.K.

Use 10 Jr 9 beam

The detailed design of horizontal shear connectors is beyond the scope of this preliminary design. However, this problem should be considered, particularly in beams subject to moment-type failures, to ensure that no separation between beam and slab occurs which would allow premature failure by lateral buckling of the compression flange of the beam.

(c) Interior Beam Cost Factors

$$C_t = w X_s \tag{3.23.13}$$

$C_t = 0.183 \times 9 =$ \$1.65
+ 0.25 cost allowance for shear connectors
1.90 \$/ft

(d) Exterior Side Beam Design and Cost Factors

Same as used for interior beam.

(e) Interior Column Design

The solution of ultimate load equations for an axially loaded steel column is quite dependent upon the properties of the assumed section. The simplest design approach is to choose a section and then investigate it to see if it is adequate. Since a 10 Jr 9 is used as the interior beam, the detailing problems of the beam-column connections are minimized, if the same section is used for both the interior beam and column.

Load acting on column

$$P = 144 q L^2 \text{ (approximate load)} \tag{4.34.3}$$

$$P = 144 \times 10 \times 49 = 70,500 \text{ lbs}$$

Ultimate capacity of the column, 10 Jr 9.

$$P = A f_{dy} \left[1 - \left(\frac{\alpha H}{5000 r_g} \sqrt{f_{dy}}\right)\right] \tag{3.22.1}$$

$$P = 2.64 \times 44,000 \left[1 - \left(\frac{0.5 \times 8.0}{5000 \times 0.48} \times \sqrt{44,000}\right)\right] = 75,500 \text{ lbs}$$

$$70,500 < 75,500 \qquad \text{Use } 10 \text{ Jr } 9$$

Normally the use of higher strength steels results in greater economy. In this example, however, the design load is such that the rolled steel shapes are at the lower limit of their applicability.

(f) Interior Column Cost Factors

$$C_t = w X_s \tag{3.23.13}$$

$$C_t = 9 \times 0.183 = 1.65 \$/\text{ft}$$

(g) Eccentrically-Loaded Side Column Design

To avoid confusion it must be understood that in the design of the eccentrically-loaded column, because of the particular structural arrangement used, the values for B and L are the reverse of those used in the design of the interior beam and column.

From the procedure outlined in Section 3.24 and Table 3-6, actual inverse column resistance function required

$$\frac{f_{dy}}{qBL} = \frac{60,000}{10 \times 6.5 \times 7.0} = 132$$

Try 10 B 11.5 since $L_{fv} = 6.60 < 7.00$ ft

Use the third form of Equation 3.24.6

$$\frac{f_{dy}}{qBL} = \frac{k_1}{L} + k_2 + 0.667\, k_3 L \qquad (3.24.6c)$$

$$\frac{f_{dy}}{qBL} = \frac{38.45}{7} + 23.37 + (0.667 \times 16.64 \times 7.00) = 106.5 < 132 \therefore \text{O.K.}$$

Use 10 B 11.5 for eccentrically-loaded side wall design.

(h) Eccentrically-Loaded Side Column Cost Factors

$$C_t = w X_s \qquad (3.23.13)$$

$$C_t = 11.5 \times 0.202 = 2.32 \text{ \$/ft}$$

(i) Concrete Masonry Wall Design

See Trial Design 3.34 F (k) for detailed analysis

Assume:

D = 6 in. RCMU	f'_{cm} = 1000 psi
L = 6.50 ft	E = 1×10^6 psi

$$e'_{cm} = \frac{1000}{1 \times 10^6} = 0.001 \text{ in./in.}$$

$$\frac{12\,L}{D} = \frac{12 \times 6.5}{6} = 13.0$$

$$R = \frac{0.001}{4}(13.0)^2 = 0.0423$$

From Table 3-66 for R = 0.0423 and 12 L/D = 13.0

$$\frac{q}{f'_{cm}} = 0.0093 \qquad q = 1000 \times 0.0093 = 9.3 \text{ psi}$$

$$9.3 > 5.0 \qquad \therefore \text{ O.K.}$$

Four corner posts of the same size used in Trial Design 4.34F are used to provide rigid supports for the masonry walls at the ends of the structure.

(j) Concrete Masonry Wall Cost Factors

4" RCMU, C_t = 1.01 \$/sq ft; 6" RCMU, C_t = 1.10 \$/sq ft

(k) Foundation Design and Unit Costs

Same as Trial Design 4.34A

(l) Required Excavation

Approximately the same as Trial Design 4.34F

(m) Entrance Way

From Table 4-7

$$C_t = \$2750$$

(n) Total Cost

Roof Slab	1.66 x 15.5 x 60.0 =	1544
Ground Slab	1.37 x 15.5 x 60.0 =	1273
Interior Beam	1.90 x 2 [59.0 - (0.83 x 8)] =	199
Exterior Beam	1.90 x 59.0 x 4 =	448

Interior and End Wall Columns	1.65 x 8.0 x 10 =	132
Exterior Columns	2.32 x 8.0 x 8 x 2 =	297
Concrete Corner Posts (from Trial Design 4.34F)	5.31 x 8.75 x 4 =	186
Exterior Side Wall 6 in. RCMU	1.10 x 59.0 x (8- 1.66) x 2 =	823
End Wall 6 in. RCMU	1.10 x 14.5 x 8.0 x 2 =	255
Excavation	0.036 x 26,800 =	965
Back Fill	0.033 x (26,800 - 8150) =	615
Haul	0.026 x 8150 =	212
Entrance Way		2750
	Total	$9699

(o) Cubicle Net Floor Area

Approximately 850 sq ft

(p) Cubicle Net Volume

Approximately 6800 cu ft

TRIAL DESIGN 4.34H

CONFIGURATION:

One story cubicle (see Figure 4-9)

STRUCTURAL SYSTEM:

Wood framing system-style A-single with one interior partition. (see Table 4-3)

DESIGN PARAMETERS:

q = 10 psi equivalent pressure including weight of roof and earth cover

L = 7.0 ft

(a) Frame Design

Assume that applicable timber properties are

$$f_f = 2000 \text{ psi}$$

$$f_{vh} = 120 \text{ psi}$$

$$f_c = 1750 \text{ psi}$$

Check for horizontal shear, assuming B = 1.25 ft

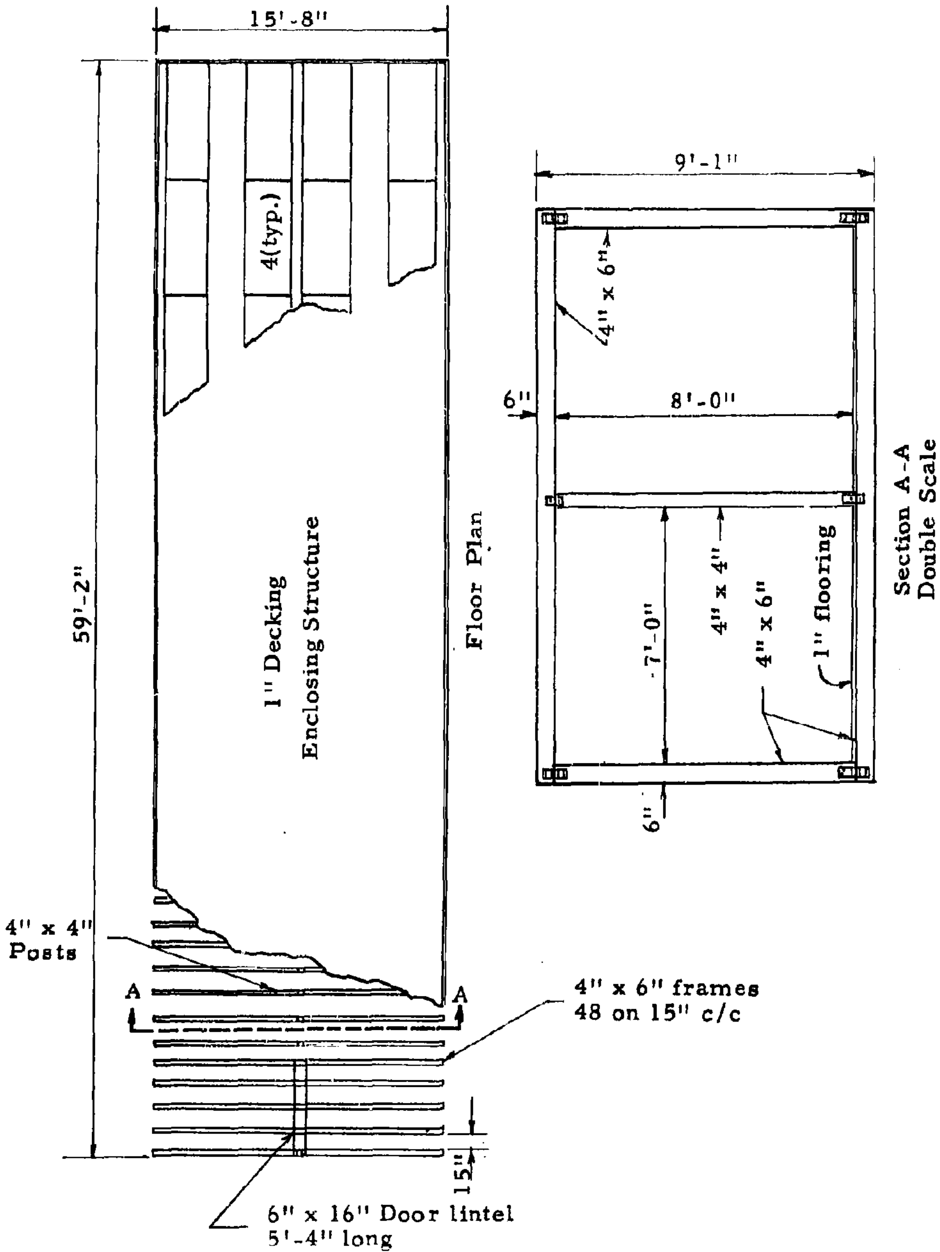


Figure 4-9

SHELTER LAYOUT TRIAL DESIGN 4.34H

Required

$$q B L = 10 \times 1.25 \times 7.0 = 87.5$$

Try 4 in. x 6 in.

From Table 3-65

$$\frac{q_v B L}{f_{vh}} = 0.755 \qquad q_v B L = 0.755 \times 120 = 90.6$$

$$90.6 > 87.5 \therefore \text{O.K. for horizontal shear}$$

Check for flexure

Required

$$q B L^2 = 10 \times 1.25 \times 49 = 613$$

Try 4 x 6

From Table 3-65

Fixed end $\dfrac{q_f B L^2}{f_f} = 0.346 \qquad q_f B L^2 = 0.346 \times 2000 = 692$

$$613 < 692 \therefore \text{O.K. for flexure}$$

Use 4 in. x 6 in. on 1.25 ft centers

$$\frac{59}{1.25} \approx 46 + 2 = 48 \text{ frames required}$$

(b) Deck Design

$$\text{span between ribs} = 1.25 - 0.33 = 0.92 \text{ ft}$$

Check for horizontal shear

Required

$$q\,B\,L = 10 \times 1.0 \times 0.92 = 9.2$$

Try d = 1/2 in.

$$\frac{q_v\,B\,L}{f_{vh}} = \frac{b\,D}{27} \qquad (3.43.10)$$

$$\frac{q_v\,B\,L}{f_{vh}} = \frac{12 \times 0.50}{27} = 0.222 \qquad q_v\,B\,L = 0.222 \times 120 = 26.6$$

$$9.2 < 26.6 \therefore \text{O.K. horizontal shear}$$

Check for moment

Required

$$q \times B \times L^2 = 10 \times 1 \times (0.92)^2 = 8.46$$

$$\frac{q_f\,B\,L^2}{f_f} = \frac{b\,D^2}{324} = \frac{12 \times (0.5)^2}{324} = 0.00926 \qquad (3.43.11)$$

$$q_f\,B\,L^2 = 0.00926 \times 2000 = 18.5$$

$$8.46 < 18.5 \therefore \text{O.K. for moment}$$

Use 3/4 in. dressed

(c) <u>Interior Beam Design (Doorway Lintel)</u>

Only one interior beam span is required to afford access between the shelter bays.

Check for horizontal shear

$$qBL = 10 \times 7.5 \times 4.68 = 351$$

Assume 6 in. x 16 in. section

From Table 3-65 and Equation 3.43.10

$$3.157 \times 120 = 379 > 351 \therefore \text{O.K. horizontal shear}$$

Check for moment

Assume simple support

$$qBL^2 = 10 \times 7.33 \times (4.67)^2 = 1599$$

for 6 in. x 16 in.

From Table 3-65 and Equation 3.43.11

$$4.078 \times 2000 = 8156 > 1605 \therefore \text{O.K. moment}$$

(d) Interior Column Design

$$P = 144 qBL$$

$$P = 144 \times 10 \times 1.25 \times 7.5 = 13.5 \text{ Kips/col.}$$

$$\frac{P}{4f_c} = \frac{13,500}{4 \times 1750} = 1.93 \text{ sq in.}$$

Use 4 in. x 4 in. post, minimum size recommended

(e) Column Area Required to Support Doorway Lintel

$$P = 144 \left[10 \times \frac{5.92}{2}\right] \times 7.5 = 32 \text{ Kips}$$

$$\frac{32,000}{4 \times 1750} = 4.5 \text{ sq in.}$$

Use 4 in. x 4 in. post, minimum size recommended

(f) Required Excavation

1) Minimum depth: $h = 3.50$ ft

2) Full burial requirement: $h = 0.00$ ft

3) Radiation requirement:

$$d_e = 3.50 + 0.02\,q \qquad (4.23.1b)$$

Use $h = d_e = 3.50 + 0.20 = 3.70$ ft

Total depth of excavation

The total depth of burial includes the earth cover h, the interior height of the structure H, the total thickness of the timber frames, top and bottom, plus the thickness of the decking.

$$z = h + H + \frac{D}{12} \qquad (4.34.1)$$

$$z = 3.70 + 8.00 + 1.25 = 12.95$$

Volume of excavation (from Equation 4.24.1)

$$\text{Vol.} = \frac{12.95}{2}\left[(59.17 \times 15.67) + (85.07 \times 41.57)\right] = 28,900 \text{ cu ft}$$

Cubicle gross volume

$$\text{Vol.} = L_T B_T \left[H + \left(\frac{D_{roof} + D_{floor}}{12}\right)\right] \qquad (4.34.2)$$

$$\text{Vol.} = 59.17 \times 15.67 \left[8.00 + \left(\frac{7.00 + 8.00}{12}\right)\right] = 8580 \text{ cu ft}$$

(g) Dead Men

Two dead men are required to carry a portion of the loading on the end walls.

$$C_T = \$300$$

(h) Entrance Way

From Table 4-4

$$C_t = \$2750$$

(i) Total Cost

Wood Frame	$0.35 \times 2.0 \times \left[(2 \times 15.5) + (2 \times 8.08)\right] \times 48 =$	1585
Frame Corner Joint	$2.00 \times 4 \times 48 =$	384
Frame Post Joints	$1.00 \times 2 \times (24 + 48) =$	144
Wood Deck Overhead and Ground Slabs	$0.35 \times 0.75 \times 15.5 \times 59.00 \times 2 =$	480
Water Proofing	$0.10 \times 15.5 \times 59.00 \times 2 =$	183
Wood Floor	$0.35 \times 1 \times 14.17 \times 59.0 =$	293
Wood Deck Sides	$0.35 \times 0.75 \times 9.25 \times 59.00 \times 2 =$	287
Water Proofing	$0.10 \times 9.25 \times 59.00 \times 2 =$	109
Wood Deck Ends	$0.35 \times 9.25 \times 15.66 \times 2 =$	101
Water Proofing	$0.10 \times 9.25 \times 15.66 \times 2 =$	29
Interior Columns	$0.35 \times 1.33 \times 8.08 \times 45 =$	170
End Columns	$0.35 \times 1.33 \times 8.08 \times 12 \times 2 =$	90
Interior Beam	$0.35 \times 8 \times 5.33 \times 2 =$	30
Excavation	$0.036 \times 28,900 =$	1040
Back Fill	$0.033 \times 20,320 =$	671
Haul	$0.026 \times 8580 =$	223
Dead Man Each End Wall	$300.00 \times 2 =$	600
Entrance Way		2750
	Total	$9169

(j) Cubicle Net Floor Area

Net floor area is $14.17 \times 59.17 = 840$ sq ft

(k) Cubicle Net Volume

Net volume is $840 \times 8.0 = 6720$ cu ft

4.4 Reinforced Concrete and Steel Arch and Cylinder

4.41 Introduction

The steel or reinforced concrete barrel arch or cylinder configurations utilize the inherent ability of such shell structures to resist compressive loadings. Major material savings are obtained, in comparison with cubicles, and some flexibility is still possible in interior layout. These structures can withstand high overpressures, and their singly-curved surfaces can be formed without too much difficulty.

Rib arch and cylinder shelter configurations are not considered as optimum underground shelter designs where only the compression failure mode is considered. The "cumulative" load effect discussed in Section 4.33 makes this configuration uneconomical in competition with the barrel shell in all pressure ranges. However, rib stiffeners can be used to strengthen conventional structures.

4.42 Layout Studies

The figures accompanying the design examples in this section show the details of the layout scheme considered as optimum in the cylinder and arch configurations. In general, the layout criteria used in this section are the same as those used in the cubicle (see Section 4.21).

4.43 Design Alternatives

(a) Reinforced Concrete Cylinder

The advantages of this type of construction is the great savings in the required volumes of structural material, as compared to the cubicle form. Its main disadvantage is the greater depth of excavation which is required. While the cylinder can have either dome or vertical end walls, economic design dictates the use of dome ends in all pressure ranges.

(b) Steel Cylinder

The design considerations discussed in connection with reinforced concrete cylinders are also applicable to steel cylinders. Both uniform thickness and corrugated steel plate are available for use.

(c) Reinforced Concrete and Steel Arch

This design has the same vertical end wall and material choices

found in the cylindrical shape. A slight economic advantage favoring the use of the vertical end wall exists in the lower pressure range. Steel arches normally will require a dead man to carry a portion of the load on the vertical end wall. While the large burial depth requirement of the cylinder is reduced by the use of an arch, it becomes necessary to provide a foundation for the structure. This frequently becomes a major item of expense, and limits the usefulness of the arch configuration. The arch shape can be considered as a compromise between the cubicle and cylinder structural systems.

4.44 Sample Analysis and Cost Evaluation

TRIAL DESIGN 4.44A

CONFIGURATION:

One story 15 ft cylinder (see Figures 4-10 and 4-11)

STRUCTURAL SYSTEM:

Reinforced concrete cylindrical shell

DESIGN PARAMETERS:

q = 100 psi equivalent pressure including weight of earth cover

S_L = 15.0 ft clear span

(a) Cylindrical Concrete Shell Design

Take D = 3.00 in., S_L = 15.50 ft (including shell thickness)

$$\frac{q_c S_L}{D} = \frac{100 \times 15.50}{3.00} = 517$$

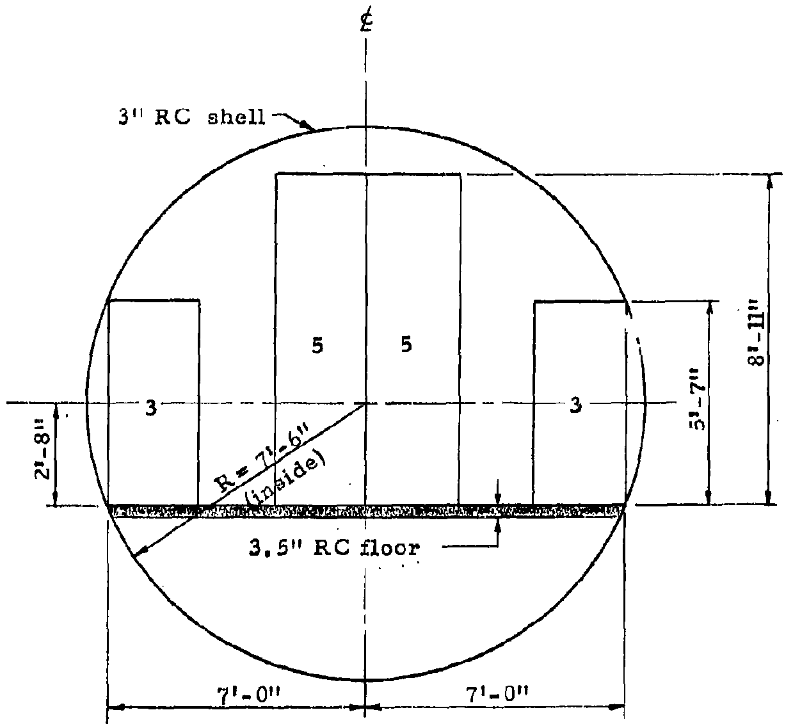


Figure 4-10

CROSS-SECTION THRU HORIZONTAL CYLINDER
SINGLE STORY, 15'-0" DIAMETER

TRIAL DESIGN 4.44A

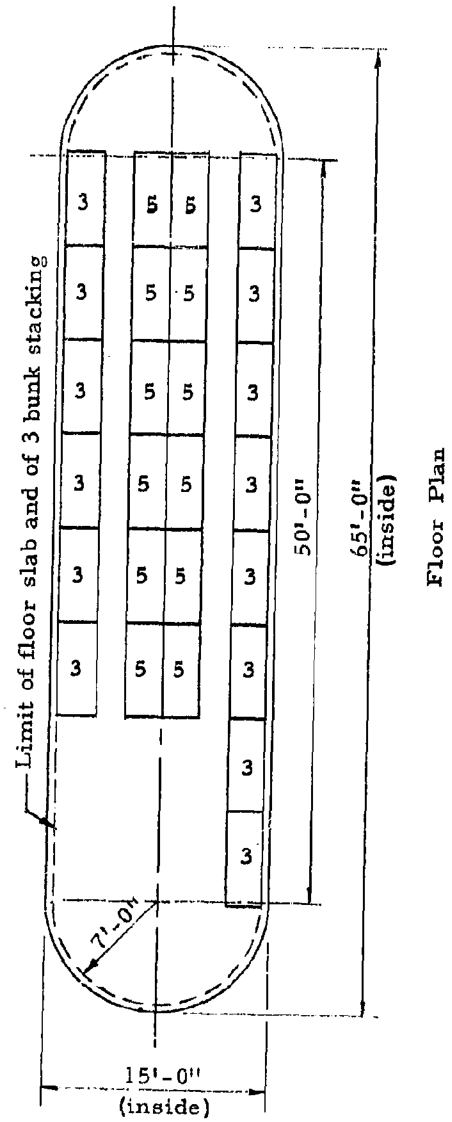


Figure 4-11

SINGLE STORY HORIZONTAL CYLINDER, 15'-0" DIAMETER

TRIAL DESIGN 4.41A

For use in Table 3-58

Assume $\phi_t = 0.50\%$, $f'_{dc} = 3750$, $f_{dy} = 60,000$ psi

$$\frac{q_c S_L}{D} = 583 > 517 \therefore \text{ O.K.}$$

A value of $f_{dy} = 44,000$ or $52,000$ psi would also be acceptable in this case, but since no differential in cost exists between the 60,000, 52,000 and 44,000 psi reinforcing rod steels, no attempt is made to use a lower strength steel.

(b) Cylindrical Shell Cost Factors

Concrete $$C_c = \left(\frac{D}{12}\right) X_c \qquad (3.37.3b)$$

$$C_c = \frac{3.00}{12} \times 1.05 = 0.27 \text{ \$/sq ft}$$

Main Steel $$C_s = \left(\frac{D\,\phi_t}{1200}\right) X_s \qquad (3.37.4a)$$

$$C_s = \frac{3.00}{12} \times 0.005 \times 85.8 = 0.11 \text{ \$/sq ft}$$

Temp. Steel $$C_{st} = \left(\frac{D\,\phi_{te}}{1200}\right) X_s \qquad (3.37.5)$$

$$C_{st} = \frac{3.00}{12} \times 0.001 \times 78.8 = 0.02 \text{ \$/sq ft}$$

Forms $$C_f = X_f \qquad (3.37.6b)$$

$$C_f = 1.40 \text{ \$/sq ft}$$

Summary

$$
\begin{aligned}
C_c &= 0.27 \\
C_s &= 0.11 \\
C_{st} &= 0.02 \\
C_f &= \underline{1.40} \\
C_t &= 1.80 \ \$/\text{sq ft}
\end{aligned}
$$

(c) Dome End Design

The design load is half that acting on the cylinder, therefore, minimum dimension will govern.

$D = 3.00$, $f'_{dc} = 2500$ psi, $f_{dy} = 60,000$ psi and $\phi_t = 0.50\%$

(d) Dome End Cost Factors

Concrete

$$C_c = \frac{3.00}{12} \times 1.00 = 0.25 \ \$/\text{sq ft} \qquad \text{from (3.35.33a)}$$

Steel

$$C_s = \frac{3.00}{12} \times \frac{1.0}{100} \times 85.8 = 0.22 \ \$/\text{sq ft} \qquad \text{from (3.35.34a)}$$

Forms

$$C_f = 1.75 \ \$/\text{sq ft} \qquad \text{from (3.35.36a)}$$

Summary

$$
\begin{aligned}
C_c &= 0.25 \\
C_s &= 0.22 \\
C_f &= \underline{1.75} \\
C_t &= 2.22 \ \$/\text{sq ft}
\end{aligned}
$$

(e) Internal Structure

The floor system is designed for 100 psi working load.

(1) Floor slab (one-way)

Assume a hinged at connection with shell. From Table 3-12, for $\theta' = 0.25$, $f'_c = 4000$ psi, and $f_{dy} = 60,000$ psi, (assumed to be optimum values) we find that $\phi_c = 0.46$ percent. Solving for d (Eq. 3.33.10) we obtain

Cost factors $d = 2.4$ in.

Use $d = 2.5$ in. and $D = 3.5$ in.

Concrete

$$C_c = \frac{3.5}{12} \times 1.21 = 0.35 \text{ \$/sq ft} \qquad \text{from (3.33.30a)}$$

Main Steel

$$C_s = \frac{1.5 \times 78.8 \times 0.46 \times 2.5}{1200} = 0.11 \text{ \$/sq ft}$$

(approximated from derivation of 3.33.30b)

Temp. Steel

$$C_{st} = \frac{3.50}{12} \times 0.001 \times 78.8 = 0.02 \text{ \$/sq ft} \qquad \text{from (3.33.30d)}$$

Forms

$$C_f = 0.92 \text{ \$/sq ft} \qquad \text{from (3.33.30e)}$$

Summary

Summary	C_c	=	0.35
	C_s	=	0.11
	C_{st}	=	0.02
	C_f	=	0.92
	C_t	=	1.40 \$/sq ft

Gross floor area = $(50 \times 14.0) + (\pi \times 7.0^2) = 700 + 154 = 854$ sq ft

(2) Floor support angle (no isolation included)

Use $\approx$ 15 lb /ft angle

Straight section = 100.0 ft   Unit cost 3.00 \$/ft

Curved section = 44.0 ft   Unit cost 5.50 \$/ft

(f) <u>Required Excavation</u>

(1) Minimum depth $h = 3.50$ ft

(2) Minimum depth of burial

From Section 4.23

$$h = 0.143 S_L$$

$$h = 0.143 \times 15.25 = 2.20 < 3.50 \therefore \text{not critical}$$

(3) Radiation required depth

$$d_e = 3.50 + 0.02 q \qquad (4.21.1b)$$

$$h = d_e = 3.50 + (0.02 \times 100) = 5.5 \text{ ft} \therefore \text{this governs}$$

Total depth of excavation

See Sample Design 4.34 B for detailed method of evaluation.

$$z = 5.50 - (1.5 \times 0.25) + 15.00 + 0.50 = 20.625 \text{ ft} \qquad \text{from } (4.34.1)$$

Following the criteria presented in Section 4.24,

Total gross volume of excavation

$$\text{Vol.} = \frac{20.625}{2}\left[(50.00 \times 15.50) + (91.25 \times 56.75)\right] \text{ (approximate)}$$

$$= 61,500 \text{ cu ft} \qquad \text{from } (4.24.1)$$

Shelter gross volume

$$\text{Vol.} = \pi \frac{(S_L)^2}{4} L_T + \frac{4\pi}{3} \frac{(S_L)^3}{8}$$

$$\text{Vol.} = \pi \times \frac{(15.5)^2}{4} \times 50 + \frac{4}{3} \times \pi \times \frac{(15.5)^3}{8} = 11,250 \text{ cu ft}$$

(g) Entrance Way

From Table 4-4

$$C_T = \$4110$$

(h) Total Cost

Cylindrical Shell	$1.80 \times 15.25 \times \pi \times 50.0 =$	4320
Dome Ends Shell	$2.22 \times (15.25)^2 \times \pi =$	1628
Internal Floor	$1.40 \times 854 =$	1194
Int. Angle Support	$(3.00 \times 100.0) + (5.50 \times 44.0) =$	542
Excavation	$0.036 \times 61,500 =$	2210
Back Fill	$0.033 \times (61,500 - 11,250) =$	1660
Haul	$0.026 \times 11,250 =$	293
Entrance Way		4110
	Total	$15,957

(i) Net Floor Area

Net floor area (head room 5'-7") is approximately 850 sq ft.

(j) Net Volume

Net volume of shelter is approximately 10,600 cu ft.

TRIAL DESIGN 4.44B

CONFIGURATION:

Two story cylinder (see Figures 4-12 and 4-13)

STRUCTURAL SYSTEM:

Steel cylinder 18.5 ft diameter

DESIGN PARAMETERS:

q = 100 psi equivalent pressure including weight of earth cover

S_L = 18.5 ft clear span

(a) Nominal Design Load on Cylinder

$$6\,q\,S_L = 6 \times 100 \times 18.5 = 11{,}100 \text{ lb/linear inch of shell}$$

(b) Steel Cylindrical Shell Design

From Table 3-8

A uniform thickness 1/4 in. plate made from a steel having a dynamic yield strength equal to 44,000 psi has a dynamic yield strength equal to 11,000 psi.

$$11{,}000 \approx 10{,}800 \;\therefore\; \text{O.K.}$$

Use 1/4 in. plate, f_{dy} = 44,000 psi

(c) Steel Cylindrical Shell Cost Factors

From Section 2.22 for 1/4 in. plate f_{dy} = 44,000 psi and single curvature from Table 2-5

$$X_s = 3.21 \text{ \$/sq ft of shell}$$

(d) Steel Dome End Design

The design load of the dome ends is only one-half that of the cylindrical sides. Therefore, 1/4 in. plate with f_{dy} = 44,000 psi will also be used for the ends.

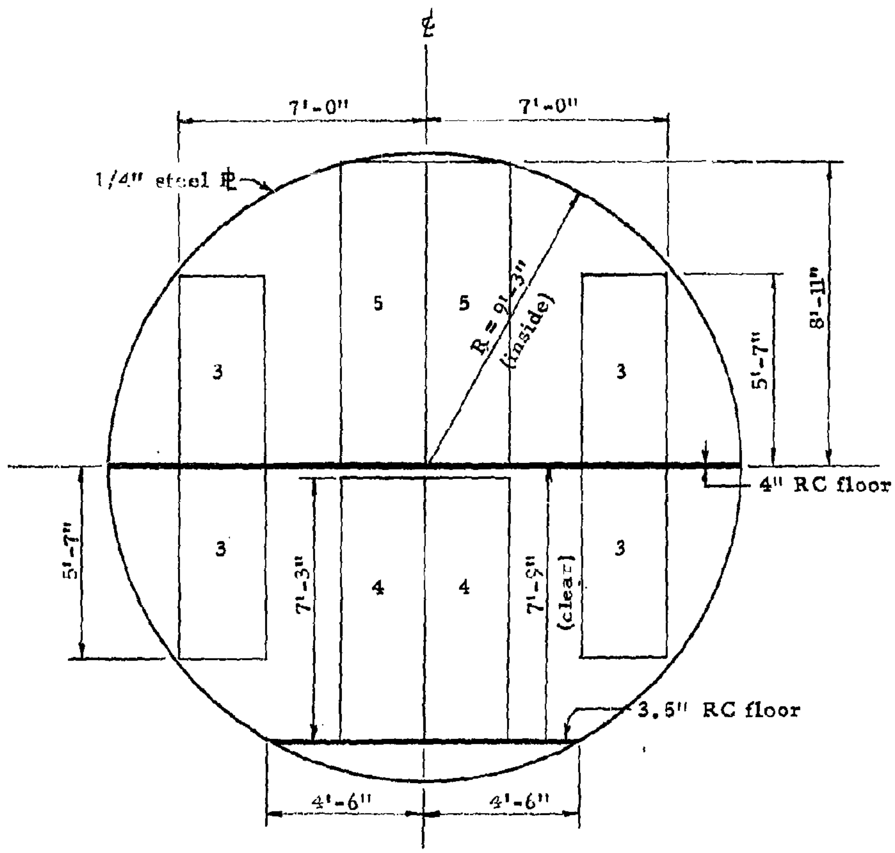


Figure 4-12
CROSS SECTION THRU 2 STORY HORIZONTAL CYLINDER
18'-6" DIAMETER
TRIAL DESIGN 4. 44B

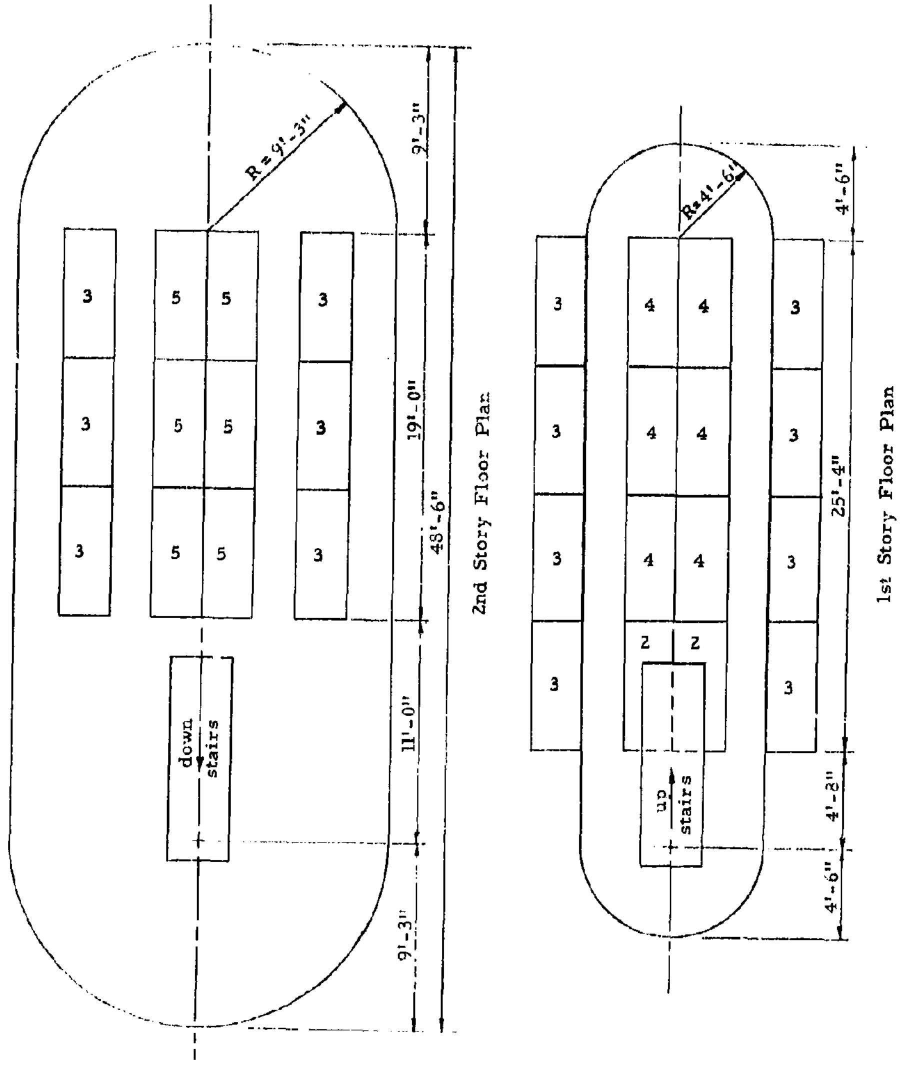


Figure 4-13
TWO STORY HORIZONTAL CYLINDER, 18'-6" DIAMETER
TRIAL DESIGN 4.44B

(e) Steel Dome End Cost Factors

From Section 2.22 for 1/4 in. plate f_{dy} = 44,000 psi and double curvature

$$X_s = 4.45 \text{ \$/sq ft of shell}$$

(f) Internal Structure Design (no shock isolation included)

Both the first and second floors of the shelters are designed to carry a 100 psi load.

Assuming a simply supported one-way slab,

First floor -

3 1/2 in. reinforced concrete slab (see Trial Design 4.44A).

ϕ_c = 0.46 percent

f'_c = 4000 psi

Use ≈ 12 lb/ft angle support

Second floor -

4 in. reinforced concrete slab (d = 3.0 in., D = 4.0 in.)

ϕ_c = 0.46 percent

f'_c = 4000 psi

Use ≈ 16 lb/ft angle support

Slab thickness is held to a minimum to provide maximum head space.

(g) Internal Structure Cost Factors (no shock isolation)

First floor slab -

Concrete

$$C_c = \frac{3.50}{12} \times 1.21 = 0.35 \text{ \$/sq ft} \qquad \text{from (3.33.30a)}$$

Main Steel $C_s = \frac{1.5 \times 78.8 \times 0.46 \times 2.5}{1200} = 0.11$ \$/sq ft (see Trial Design 4.44A)

Temperature Steel $C_{st} = \frac{3.50}{12} \times 0.001 \times 78.8 = 0.02$ \$/sq ft from (3.33.30d)

Forms $C_f = 0.92$ from (3.33.30e)

Summary

$$C_c = 0.35$$
$$C_s = 0.11$$
$$C_{st} = 0.02$$
$$C_f = \underline{0.92}$$
$$C_t = 1.40 \text{ \$/sq ft}$$

Second floor slab-

Concrete $C_c = \frac{4.00}{12} \times 1.21 = 0.40$ \$/sq ft from (3.33.30a)

Main Steel $C_s = \frac{1.5 \times 78.8 \times 0.46 \times 4.0}{1200} = 0.18$ \$/sq ft

Temperature Steel $C_{st} = \frac{5.00}{12} \times 0.001 \times 78.8 = 0.03$ \$/sq ft from (3.33.30d)

Forms $C_f = 0.94$ \$/sq ft from (3.33.30c)

Summary

$$C_c = 0.40$$
$$C_s = 0.18$$
$$C_{st} = 0.03$$
$$C_f = \underline{0.93}$$
$$C_t = 1.54 \text{ \$/sq ft}$$

(h) Angle Support Cost Factors

From Section 2.29

First floor

Straight $X_s = 2.50$ \$/ft

Second floor

Straight $X_s = 2.50$ \$/ft

Curved X_s = 5.00 \$/ft   Curved X_s = 5.00 \$/ft

Stairway from first to second floor

From Section 2.29

$$C_T = \$600$$

(i) Required Excavation

(1) Minimum cover $h = 3.5$ ft

(2) Cover required for full burial

$$h = 0.143 S_L = 0.143 \times 18.5 = 2.64 \text{ ft}$$

(3) Radiation burial requirement

$$d_e = 3.5 + 0.02q \qquad (4.21.1b)$$

$$h = d_e = 3.5 + (0.02 \times 100) = 5.5 \text{ ft}$$

Total depth of excavation

$$z = 18.50 + 5.50 = 24.00 \text{ ft} \qquad \text{from } (4.34.1)$$

Total volume of excavation (see Section 4.24)

$$\text{Volume} = \frac{24.00}{2}\left[(30.0 \times 18.5) + (78.0 \times 66.5)\right] \text{(approximately)}$$
$$= 68,900 \text{ cu ft} \qquad \text{from } (4.24.1)$$

Shelter gross volume

$$\text{Volume} = 3.14 \times \frac{(18.50)^2}{4} \times 30.0 + \frac{4}{3} \times 3.14 \times \frac{(18.5)^3}{8} = 11,380 \text{ cu ft}$$

(j) Entrance Way

From Table 4-4

$$C_t = \$4050$$

(k) Total Cost

Steel Cylindrical Shell	$3.21 \times \pi \times 18.5 \times 30.0 =$	5600
Steel Dome Ends	$4.45 \times \pi \times 18.5^2 =$	4790
First Floor Slab	$1.40 \times \left[(9.0 \times 30.0) + (4.5^2 \times \pi)\right] =$	466
Second Floor Slab	$1.56 \times \left[(30.0 \times 18.5) + (\frac{18.5^2}{4} \times \pi)\right] =$	1269
First Floor Angle Support	$(2.50 \times 2 \times 30.0) + (5.00 \times 28.3) =$	292
Second Floor Angle Support	$(2.50 \times 2 \times 30) + (5.00 \times 58.0) =$	440
Stairs		600
Excavation	$0.036 \times 68,900 =$	2400

Back Fill	0.033 x (68,900 - 11,380) =	1900
Haul	0.026 x 11,380 =	296
Entrance Way		4050
	Total	22,183

(l) Net Floor Area

Net floor area (headroom ≥ 5'-7") is approximately 850 sq ft.

(m) Net Volume

Net volume of shelter is approximately 11,380 cu ft.

TRIAL DESIGN 4.44C

CONFIGURATION:

One story 18.5' diameter arch (see Figures 4-14 and 4-15)

STRUCTURAL SYSTEM:

Steel cylindrical barrel arch (corrugated) with vertical ends

DESIGN PARAMETERS:

q = 25 psi equivalent pressure including weight of earth cover

S_L = 18.5 ft

(a) Nominal Design Load

$$6\,qL = 6 \times 25 \times 18.5 = 2770 \text{ lb /in. of shell}$$

(b) Corrugated Steel Shell Design

From Table 3-7

For No. 12 gage $\quad f_{dy} = 44{,}000$ psi

$$P = 5700 \text{ lb /in.}$$

Multiplying this value by the seam reduction factor discussed in Section 3.26,

$$5700 \times 0.70 = 3990 > 2770 \therefore \text{O.K.}$$

Use 12 gage corrugation

(c) Corrugated Shell Cost Factor

From Section 2.22 Table 2-6

$$C_t = 2.84 \text{ \$/sq ft}$$

(d) End Wall Design

End walls for cylinder and arch configurations are designed as a

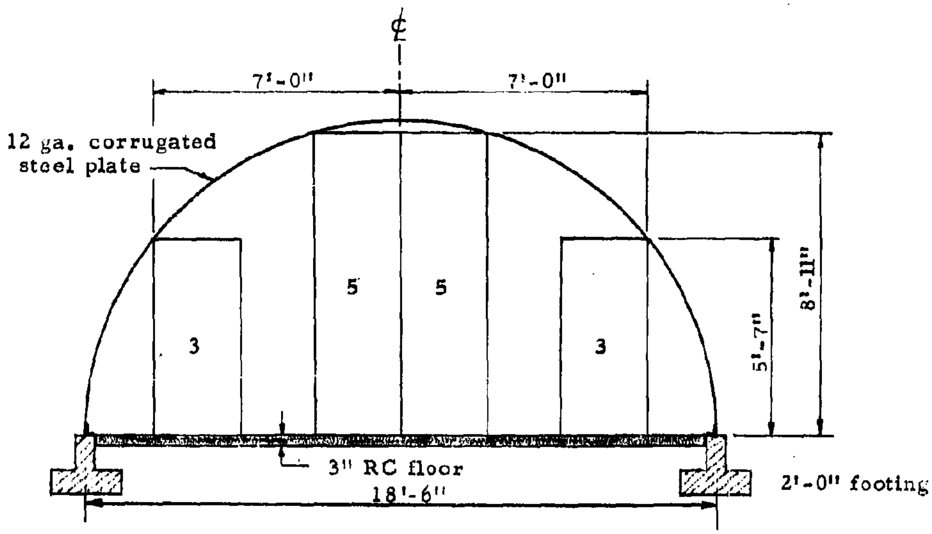


Figure 4-14
ONE STORY ARCH, 18'-6" DIAMETER
TRIAL DESIGN 4.44C

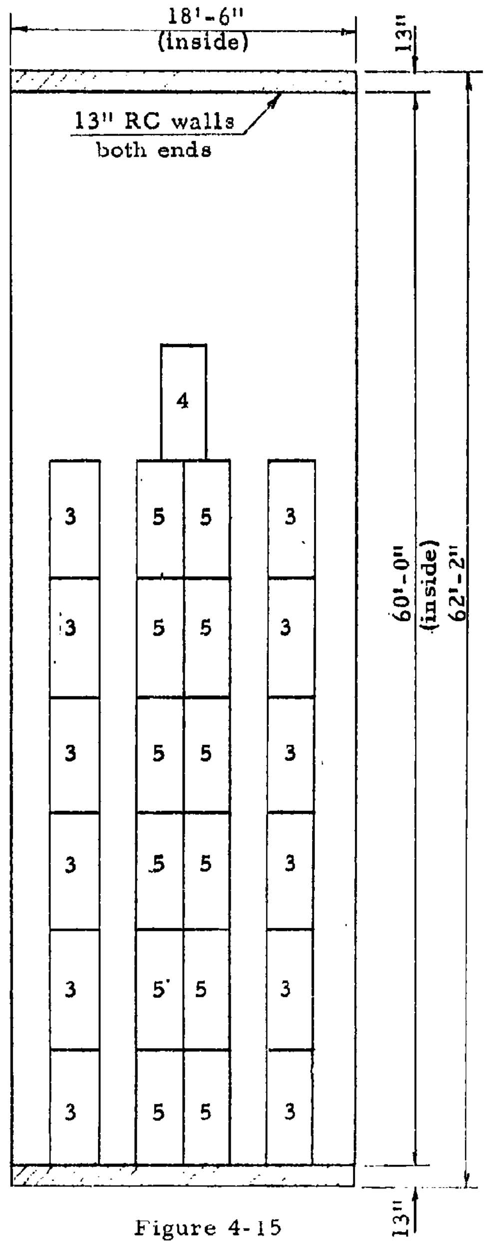


Figure 4-15
FLOOR PLAN OF ONE STORY ARCH, 18'-6" DIAMETER
TRIAL DESIGN 4.44C

two-way simply supported reinforced concrete slab (α = 1) having a span length equal to S_L in the case of cylinders and $S_L/2$ for arches. The unit loading is taken as one-half that load acting radially on the shell.

$$q_{\text{end wall}} = \frac{q_{\text{shell}}}{2}$$

$$q = \frac{25 \text{ psi}}{2} = 12.5 \text{ psi}$$

$$L = \frac{18.5}{2} = 9.25 \text{ ft}$$

Check to see if minimum dimension wall D = 6.00 in., ϕ_t = 0.5 percent is adequate. From Table 3-16, for f_{dy} = 60,000 psi, f'_c = 4000 psi, ϕ_v = 0, and θ' = 0.25, we find $\phi_{Sc} = \phi_{Lc}$ = 0.091 percent for $q_f = q_{sc}$.

Also, for q = 12.5 psi and $L_S = L_L$ = 9.25 ft,

$$d = 11.5 \text{ in.}$$

$$D = 13.0 \text{ in.}$$

We will use ϕ_t = 0.50 percent, hence actual $\phi_{Sc} = \frac{0.50}{2 \times 0.9} = 0.28$.

This simply means that the slab has additional strength in the tensile flexural mode but will still (according to the fundamental analysis) be limited by its diagonal tension resistance.

Dead men are required to carry a major portion of the reaction of the end wall, since the corrugated steel cylinder cannot be expected to withstand large compressive loads directed along its longitudinal axis.

(e) End Wall Cost

Concrete	$C_c = \frac{13.0}{12} \times 1.21 = 1.31$ \$/sq ft	from (3.33.30a)
Main Steel	$C_s = \frac{2 \times 78.8 \times 0.50 \times 13.0}{1200} = 0.85$ \$/sq ft	
	(approximate-provides for 0.25 percent steel in each face in each direction, without allowance for end anchorage)	
Temperature Steel	None required	
Forms	C_f = 1.00 \$/sq ft	from (3.35.36a)

Summary

$$C_c = 1.31$$
$$C_s = 0.85$$
$$C_f = \underline{1.00}$$
$$C_t = 3.16 \text{ \$/sq ft}$$

(f) <u>Foundation Design (footings)</u>

From Section (a)

$$P = 2770 \times 12 = 33.2 \text{ kips/ft of shell}$$

Assume footing width = 2.0 ft (from Table 3-59 allowable = 24 kips/sq ft)

$$\frac{P}{L_{footing}} = \frac{3.32}{2.0} = 16.6 < 24 \therefore \text{O.K.}$$

Assume 6.00 in. foundation side wall beneath arch side-walls.

$$\frac{D_{wall}}{L} = \frac{6.00}{2.0} = 3$$

From Table 3-60 for D/L = 3.0, f'_c = 2000 psi and P/L = 16,600 lb

$$\frac{d_{footing}}{L} = 2.20 \times 2.0 = 4.40 \text{ in.}$$

Use d = 4.40 in.

L = 2.0 ft

Use minimum depth of footing slab D = 8.00 in. (Section 4.22)

For

$$\frac{d_{footing}}{12L - D_{wall}} = \frac{4.40}{(24-6)} = 0.244,$$

f_{dy} = 60,000 psi and P/L = 16,600 lb

From Table 3-61

$$\phi_c = 0.40 \text{ percent}$$

(g) <u>Foundation Cost Factors</u>

(1) Footing

Concrete
$$C_c = \left(\frac{D}{12}\right) X_c \tag{3.39.9}$$

$$C_c = \frac{8.00}{12} \times 0.95 = 0.64 \text{ \$/sq ft}$$

Main Steel $$C_s = \frac{d}{12}\left(\frac{\phi_c}{100}\right) X_s \qquad (3.39.10a)$$

$$C_s = \frac{4.40}{12} \times 0.004 \times 78.8 = 0.12 \text{ \$/sq ft}$$

Temperature Steel $$C_{st} = \frac{D}{12}\left(\frac{\phi_t}{100}\right) X_s \qquad (3.39.11)$$

$$C_{st} = \frac{8.00}{12} \times 0.001 \times 78.8 = 0.06 \text{ \$/sq ft}$$

Forms $$C_f = X_f \qquad (3.39.12)$$

$$C_f = 0.75 \text{ \$/sq ft}$$

Summary

$C_c = 0.64$

$C_s = 0.12$

$C_{st} = 0.06$

$C_f = \underline{0.75}$

$C_t = 1.57$ \$/sq ft

(2) Side foundation wall

Concrete $C_c = \frac{6.00}{12} \times 1.00 = 0.50$ \$/sq ft from (3.35.33a)

Main Steel $C_s = \frac{6.00}{12} \times 0.005 \times 78.8 = 0.20$ \$/sq ft from (3.35.34c)

Temperature Steel $C_{st} = \frac{6.00}{12} \times 0.001 \times 78.8 = 0.04$ \$/sq ft from (3.35.35)

Forms $C_f = 1.00$ \$/sq ft from (3.35.36)

Summary

$C_c = 0.50$

$C_s = 0.20$

$C_{st} = 0.04$

$C_f = \underline{1.00}$

$C_t = 1.74$ \$/sq ft

(3) Floor slab

Same as Trial Design 4.34B-3 in. mesh-reinforced slab.

$$C_t = 0.91 \text{ \$/sq ft}$$

(h) Required Excavation

(1) Minimum depth $h = 3.5$ ft

(2) Minimum depth required for full burial

$$h = 0.143\ S_L$$

$$h = 0.143 \times 18.5 = 2.64 \text{ ft} < 3.5 \text{ ft} \quad \therefore \text{ not critical}$$

(3) Radiation required depth

$$d_e = 3.5 + 0.02q \qquad (4.21.1b)$$

$$h = d_e = 3.5 + 0.02 \times 25 = 4.0 \text{ ft} > 3.5 \text{ ft} \quad \therefore \text{ this governs}$$

Total depth of excavation

$$z = 4.0 + 9.25 \times 0.25 = 13.50 \text{ ft} \qquad \text{from (4.34.1)}$$

Following the criteria presented in Section 4.24,

Total volume of excavation

$$\text{Volume} = \frac{13.50}{2}\left[(62.17 \times 18.5) + (89.17 \times 45.50)\right] \qquad \text{from (4.24.1)}$$
$$= 35,100 \text{ cu ft}$$

Shelter gross volume

$$\frac{18.5^2}{4} \times 62.17 \times \frac{\pi}{2} = 8350 \text{ cu ft}$$

Trenching volume and cost factor

width of footing trench = 3.0 ft

depth of footing trench = 1.67 ft

$$C_t = 0.10 \times 1.67 \times 3.0 = 0.50 \text{ \$/ft of trench}$$

(i) Entrance Way

From Table 4-4

$$C_T = \$3050$$

(j) Total Cost

Cylindrical Shell	$2.84 \times \frac{18.5}{2} \times \pi \times 60.0 =$	4950
End Walls	$3.16 \times \pi \times \frac{(18.5)^2}{8} \times 2 =$	850

Floor Slab	$0.91 \times 60.0 \times 18.5 =$	1010
Footing	$1.57 \times \left[(2 \times 60.0 \times 2) + (2 \times 18.5 \times 2)\right] =$	493
Foundation Wall	$1.74 \times 1.0 \times \left[(2 \times 60.0) + (2 \times 18.5)\right] =$	273
Dead Men	$400 \times 2 =$	800
Trenching	$0.50 \times 157 =$	79
Excavation	$0.036 \times 35,100 =$	1264
Back Fill	$0.033 \times (35,100 - 8350) =$	883
Haul	$0.026 \times 8350 =$	217
Entrance Way		3050
	Total	$13,869

(k) Net Floor Area

Net floor area (headroom $\geq$ 5'-7") is approximately 885 sq ft.

(l) Net Volume

Net volume of shelter is approximately 8050 cu ft.

TRIAL DESIGN 4.44D

CONFIGURATION:

One story 18.5 ft inside diameter arch (see Figures 4-14 and 4-15)

STRUCTURAL SYSTEM:

Reinforced concrete cylindrical barrel arch with vertical end wall.

DESIGN PARAMETERS:

q = 25 psi equivalent pressure including weight of earth cover

S_L = 18.75 ft, including shell thickness

(a) Concrete Shell Design

Take D = 3.00 in. and check for adequacy.

$$\frac{q_c S_L}{D} = \frac{25 \times 18.75}{3.00} = 156$$

For use in Table 3-58

Assume f_{dy} = 60,000 psi, f'_{dc} = 2500 psi and ϕ_t = 0.5 percent

From Section 3.37, Table 3-58

$$\frac{q_c S_L}{D} = 405 > 156 \therefore \text{O.K.}$$

It is obvious from Table 3-58 a 44,000 psi steel would also be adequate

but there is no cost advantage in its use. (see Table 2-7)

(b) Concrete Shell Cost Factors

Concrete $C_c = \frac{3.00}{12} \times 1.00 = 0.25$ \$/sq ft from (3.37.3b)

Main Steel $C_s = \frac{3.00}{12} \times 0.005 \times 85.8 = 0.11$\$/sq ft from (3.37.4a)

Temperature Steel $C_{st} = \frac{3.00}{12} \times 0.001 \times 85.8 = 0.02$ \$/sq ft from (3.37.5)

Forms $C_f = 1.40$ \$/sq ft from (3.37.6b)

Summary
$C_c = 0.25$
$C_s = 0.11$
$C_{st} = 0.02$
$C_f = \underline{1.05}$
$C_t = 1.43$ \$/sq ft

The total cost of the rest of the shelter structure is approximately equal to that shown in Trial Design 4.44C. The slight increase in overall shelter dimensions caused by the 3.00 in. concrete shell is balanced with regard to excavation costs by the decreased depth of burial afforded by the radiation protection given by the shell.

The support for end wall loading afforded by the 3.00 in. shell walls permits the elimination of dead men to support the vertical end walls.

(c) Total Cost

Cylindrical Shell	$1.43 \times \frac{18.75}{2} \times \pi \times 60.00 =$	2530
End Walls	$3.16 \times \pi \times \frac{18.5^2}{4} =$	850
Floor Slab	$0.91 \times 18.5 \times 60.0 =$	1010
Footing	$1.57 \times \left[(2 \times 60.0 \times 2) + (2 \times 18.5 \times 2)\right] =$	493
Foundation Wall	$1.74 \times 1.0 \times 157 =$	273
Trenching	$0.50 \times 157 =$	79
Excavation	$0.036 \times 35,450 =$	1276
Back Fill	$0.033 \times 26,640 =$	880

Haul	0.026 x 8810 =	229
Entrance Way		3050
	Total	$10,670

(d) Net Floor Area

Net floor area (headroom $\geq$ 5'-7") is 885 sq ft.

(e) Net Volume

Net volume of shelter is approximately 8050 cu ft.

4.5 Reinforced Concrete and Steel Dome and Sphere

4.51 Introduction

The two-way resistance induced by the double curvature of dome and sphere shells permits design thicknesses of only half those required for cylindrical or single-curvature shells with the same design load. This property makes the double curvature shell particularly suitable for use in the very high overpressure ranges. As explained in Sections 3.37 and 3.38, it is assumed [1,3] that lateral soil restraint will preclude any buckling of the shells prior to yielding in a compressive failure mode. The major disadvantages of the double-curvature shell structures are their high forming costs and the relatively large volume of unusable space.

4.52 Layout Studies

The figures accompanying the design examples in this section show the details of the recommended layouts in the dome and sphere configurations. The layout criteria used in this section are the same as those used in the cubicle designs, and are described in Section 4.21.

4.53 Design Alternatives

(a) Reinforced Concrete and Steel Sphere

The sphere is the most efficient structural shape for resisting uniform radial loading. Spheres requiring a minimum of structural material can carry very heavy loads and, with the assumption of uniform radial loading prior to failure, eliminate any requirement for separate and costly foundation designs. The disadvantages of this type of construction are found in the large depths of excavation which are required, and in the high cost of forming double curvature shells. The spherical shell can be constructed from either reinforced concrete or uniform thickness steel plate.

(b) Reinforced Concrete and Steel Dome

The dome configuration permits a large reduction in burial depth requirements, since footings are substituted for the lower half of the sphere. As a consequence, the overpressure levels at which the dome can be effectively employed are controlled by the bearing capacity of the soil beneath the footing. In general, the dome configuration combines many of the disadvantages of the cubicle and shell structures. It is not considered a likely candidate for an underground shelter configuration.

4.54 Sample Analysis and Cost Evaluation

TRIAL DESIGN 4.54A

CONFIGURATION:

Three story sphere (see Figure 4-16 and 4-19)

STRUCTURAL SYSTEM:

Reinforced concrete sphere, 28.0 ft inside diameter

DESIGN PARAMETERS:

q = 325 psi equivalent pressure including weight of earth cover

S_L = 28.33 ft including shell thickness

(a) Reinforced Concrete Shell Design

When concrete elements are acting in direct compression, the most economical designs employ higher strength materials with minimum quantities of steel.

For use in Table 3-58, for doubly curved shells, assume $f_{dy} = 75,000$ psi, $f'_{dc} = 7500$ psi and $\phi_t = 0.50$ percent.

From Section 3.38, Table 3-58

$$\frac{q_c S_L}{D} = 2256$$

$$\frac{325 \times 28.33}{2256} = D_{required} = 4.08 \text{ in.}$$

Use D = 4.5 in. $\phi_t = 0.50$ percent.

In this particular case it would be possible to increase ϕ_t slightly and thereby reduce D slightly.

Assume

$f_{dy} = 75,000$ psi, $f'_{dc} = 7500$ psi and $\phi_t = 0.70$ percent

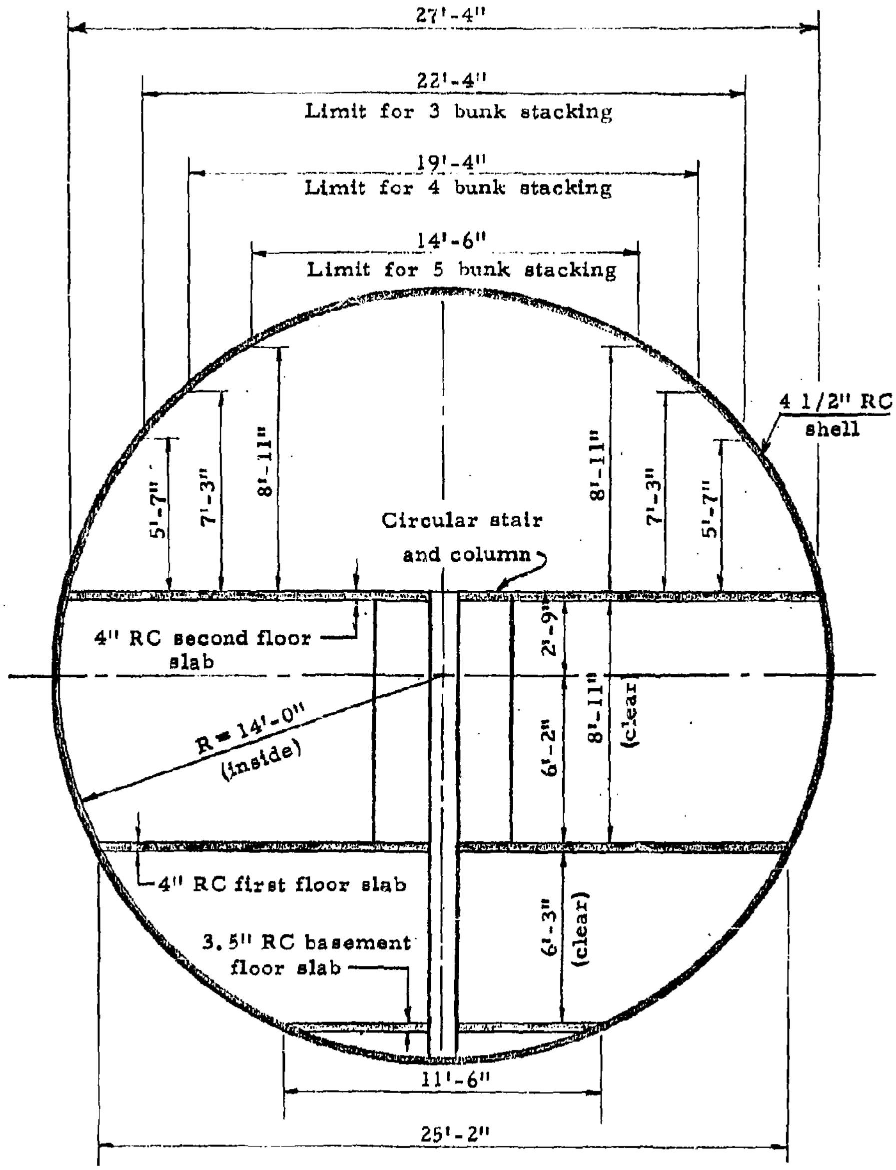


Figure 4-16

SECTIONAL ELEVATION OF SPHERE, 28'-0" DIAMETER

TRIAL DESIGN 4.54A

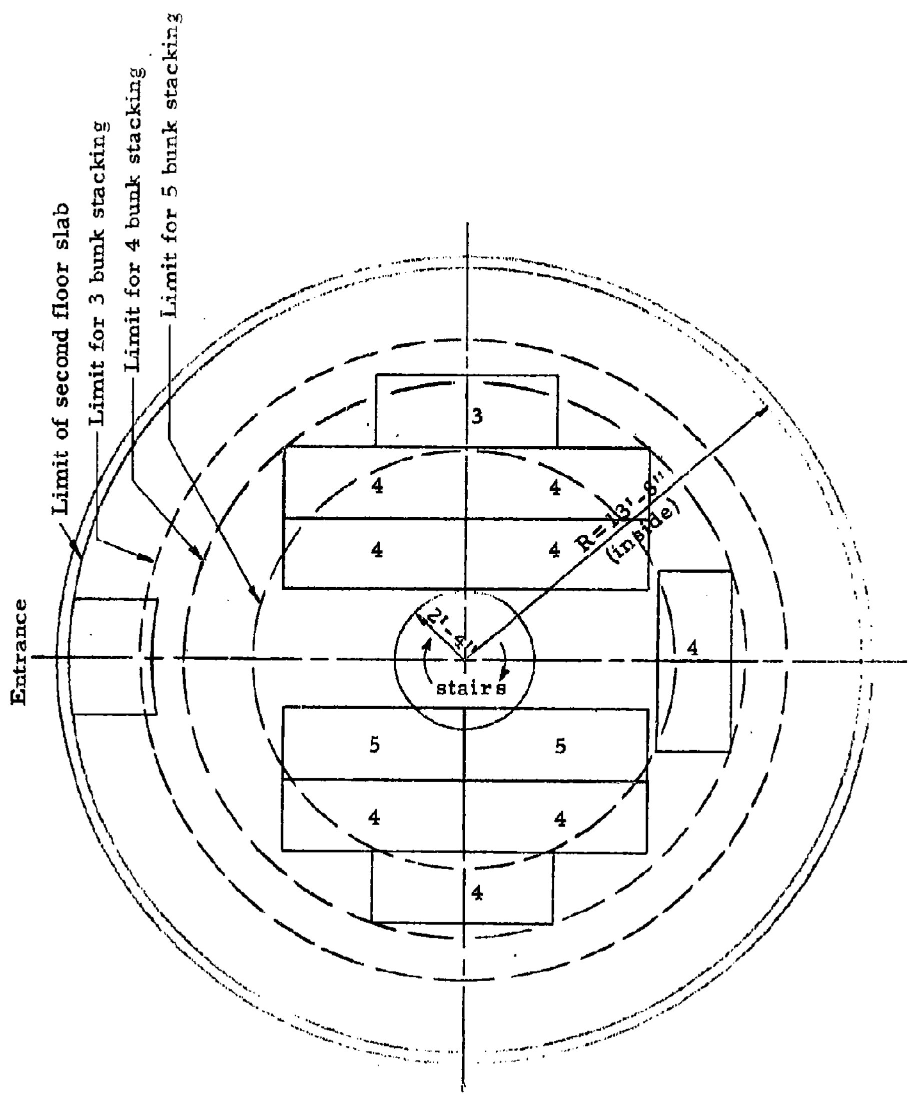


Figure 4-17
SECOND FLOOR PLAN OF SPHERE, 28'-0" DIAMETER
TRIAL DESIGN 4.54A

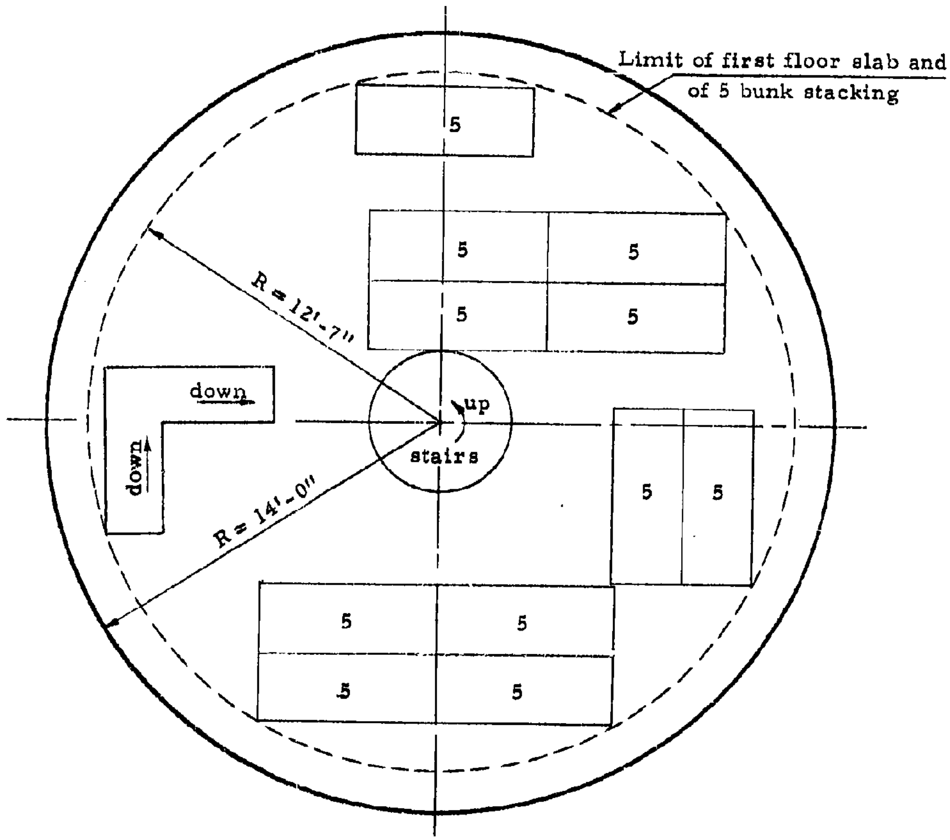


Figure 4-18
FIRST FLOOR PLAN OF SPHERE, 28'-0" DIAMETER
TRIAL DESIGN 4.54A

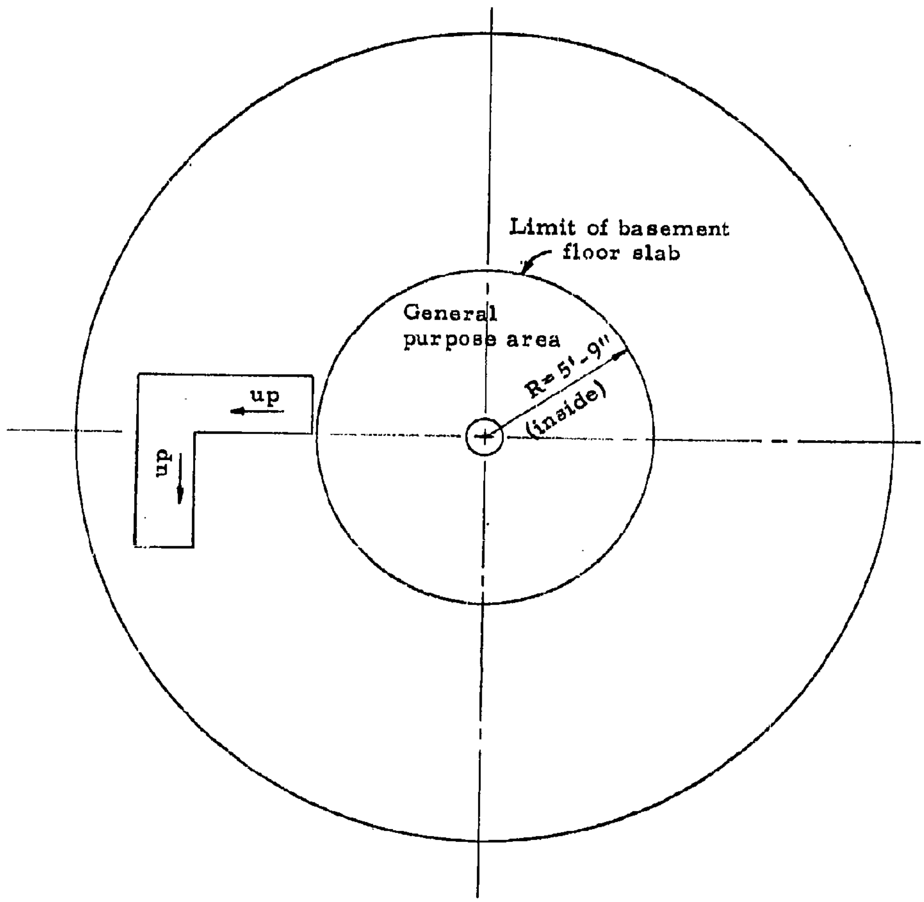


Figure 4-19
BASEMENT PLAN OF SPHERE, 28'-0" DIAMETER
TRIAL DESIGN 4.54A

$$\frac{q_c S_L}{D} = 0.284 f'_{dc} + 0.003333\, \phi_t f_{dy} \qquad (3.38.1)$$

$$\frac{q_c S_L}{D} = (0.284 \times 7500) + (0.003333 \times 0.70 \times 75,000) = 2305$$

$$\frac{325 \times 28.33}{2305} = D_{required} = 3.99 \text{ in.}$$

Use D = 4.00 in. and ϕ_t = 0.70 percent.

(b) Reinforced Concrete Shell Unit Costs

Costs will be calculated for D = 4.5 in. and ϕ_t = 0.50 percent.

Concrete $\quad C_c = \frac{4.50}{12} \times 1.30 = 0.49$ \$/sq ft from (3.37.3b)

Steel
$$C_s = 2\left(\frac{D\,\phi_t}{1200}\right) X_s \qquad (3.38.2)$$

$$C_s = 2 \times \frac{4.50 \times 0.50}{1200} \times 100.5 = 0.38 \text{ \$/sq ft}$$

Forms $\quad C_f = 1.75$ \$/sq ft from (3.37.6b)

Summary
$$C_c = 0.49$$
$$C_s = 0.38$$
$$C_f = \underline{1.75}$$
$$C_t = 2.62 \text{ \$/sq ft}$$

The second design of D = 4.00 in. and ϕ_t = 0.70 percent would result in 0.04 \$/sq ft higher costs.

(c) Internal Structure Design (no shock isolation included)

All three floors are designed to carry a 100 psi working load. The floors are assumed to be simply-supported slabs with L = 11.0 ft. Proceeding as outlined for Design Example 4.44A,we obtain D = 3.5 in. All floor slabs will be supported with curved angle weighing 12 lb/ft.

Central column support

D = 12 in. circular column

ϕ_t = 0.50 percent

$$f'_{dc} = 2500 \text{ psi}$$

$$f_{dy} = 60,000 \text{ psi}$$

A 12 in. circular column is larger than required to carry floor loads, but is considered necessary to provide for adequate anchorage of circular stairs.

(d) Internal Structure Unit Costs

(1) Floor slabs. From Design Example 4.44A we obtain

$$C_t = 1.40 \text{ \$/sq ft}$$

(2) Angle support for all floor slabs. From Section 2.29

Curved Angle $\quad C_t = X_s = 5.00 \text{ \$/ft}$

(3) Central column support

Concrete
$$C_c = \frac{A}{144} X_c \qquad (3.32.4a)$$

$$C_c = \frac{(6.00)^2 \times \pi}{144} \times 1.00 = 0.79 \text{ \$/ft}$$

Main Steel
$$C_s = \frac{A}{144}\left(\frac{\phi_t}{100}\right) X_s \qquad (3.32.5a)$$

$$C_s = \frac{113}{144} \times 0.005 \times 78.8 = 0.31 \text{ \$/ft}$$

Tie Steel
$$C_{st} = \frac{A}{144}\left(\frac{\phi_{te}}{100}\right) X_s$$

$$C_{st} = \frac{113}{144} \times 0.001 \times 78.8 = 0.06 \text{ \$/ft}$$

Forms
$$C_f = X_f P_r \qquad (3.32.7a)$$

$$C_f = 1.10 \times 1.00 \times 3.14 = 3.46 \text{ \$/ft}$$

Summary

$$C_c = 0.79$$

$$C_s = 0.31$$

$$C_{te} = 0.06$$

$$C_f = \underline{3.46}$$

$$C_t = 4.62 \text{ \$/ft}$$

(4) Circular stairs

One floor at $750 per floor

(5) Rectangular stairs

One floor at $600 per floor

(e) Required Excavation

(1) Minimum depth $h \doteq 3.5$ ft

(2) Cover required for full burial

$$h = 0.125\ S_L = 0.125 \times 28.75 = 3.59 \text{ ft}$$

(3) Radiation burial requirement

$$d_e = 3.5 + 0.02q$$

$$d_e = 3.5 + (0.02 \times 325) = 10.0 \text{ ft}$$

$$d_e = 10.0 - (1.5 \times 0.33) = 9.5 \text{ ft}$$

Total depth of excavation

$$z = 28.75 + 9.50 \doteq 38.25 \text{ ft}$$

Total volume of excavation

Assume the slope of the excavation is the frustum of a cone having a 1:1 slope with the lower base diameter equal to 12.0 ft and the base diameter of the ground surface equal to 88.50 ft. Following the criteria presented in Section 4.24 the volume of the excavation is taken as

$$\text{Volume} = \frac{z}{3}\left(A_1 + A_2 + \sqrt{A_1 A_2}\right)$$

$$\text{Volume} = \frac{38.25}{3}\left[\left(\pi \times \frac{12.00^2}{4}\right) + \left(\pi \times \frac{88.50^2}{4}\right) + \sqrt{113 \times 6150}\right] = 90,500 \text{ cu ft}$$

Volume of structure

$$\text{Volume} = \frac{4\pi}{3}\left(\frac{S_L}{2}\right)^3 = \frac{4}{3} \times 3.14 \times \frac{28.75^3}{8} = 12,430 \text{ cu ft}$$

(f) Entrance Way

From Table 4-4

$$C_T = \$6690$$

(g) Total Cost

Reinforced Concrete Spherical Shell	$2.62 \times 4 \times \pi \times \left(\frac{28.38}{2}\right)^2 =$	6630
First Floor Slab	$1.40 \times \pi \times \left(\frac{24.83}{2}\right)^2 =$	680
Second Floor Slab	$1.40 \times \pi \times \left(\frac{27.33}{2}\right)^2 =$	823
Basement Floor Slab	$1.40 \times \pi \times \left(\frac{11.5}{2}\right)^2 =$	145
Angle Slab Support	$5.00 \times (24.83 + 27.33 + 11.5) \times \pi =$	1000
Center Column	$4.62 \times 17.08 =$	79
Stairs	$750 + 600 =$	1350
Excavation	$0.036 \times 90,500 =$	3260
Back Fill	$0.033 \times (90,500 - 12,430) =$	2580
Haul	$0.026 \times 12,430 =$	323
Entrance Way		6690
	Total	\$23,560

The excavation cost could probably be reduced by using techniques better suited to deep excavations.

(h) Net Floor Area

Net floor area (headroom $\geq$ 5'-7'') is approximately 975 sq ft.

(i) Net Volume

Net volume of shelter is 11,500 cu ft.

TRIAL DESIGN 4.54B

CONFIGURATION:

One story dome (see Figures 4-20 and 4-21)

STRUCTURAL SYSTEM:

Steel dome 36.0 ft inside diameter

DESIGN PARAMETERS:

q = 100 psi equivalent pressure including weight of earth cover

S_L = 36.0 ft

(a) Nominal Design Load on Dome

$$3\,q\,S_L = 3 \times 100 \times 36 = 10,800 \text{ lb/in. of steel}$$

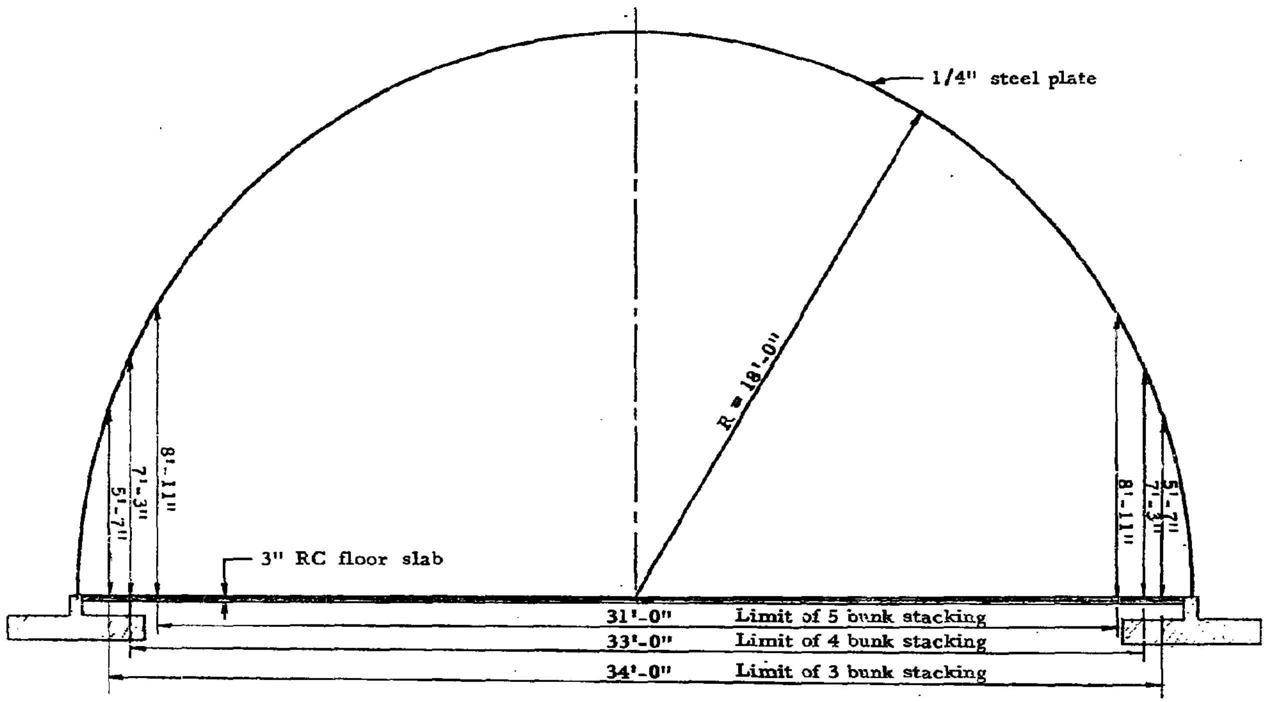


Figure 4-20
ONE STORY DOME, 36'-0" DIAMETER
TRIAL DESIGN 4.54B

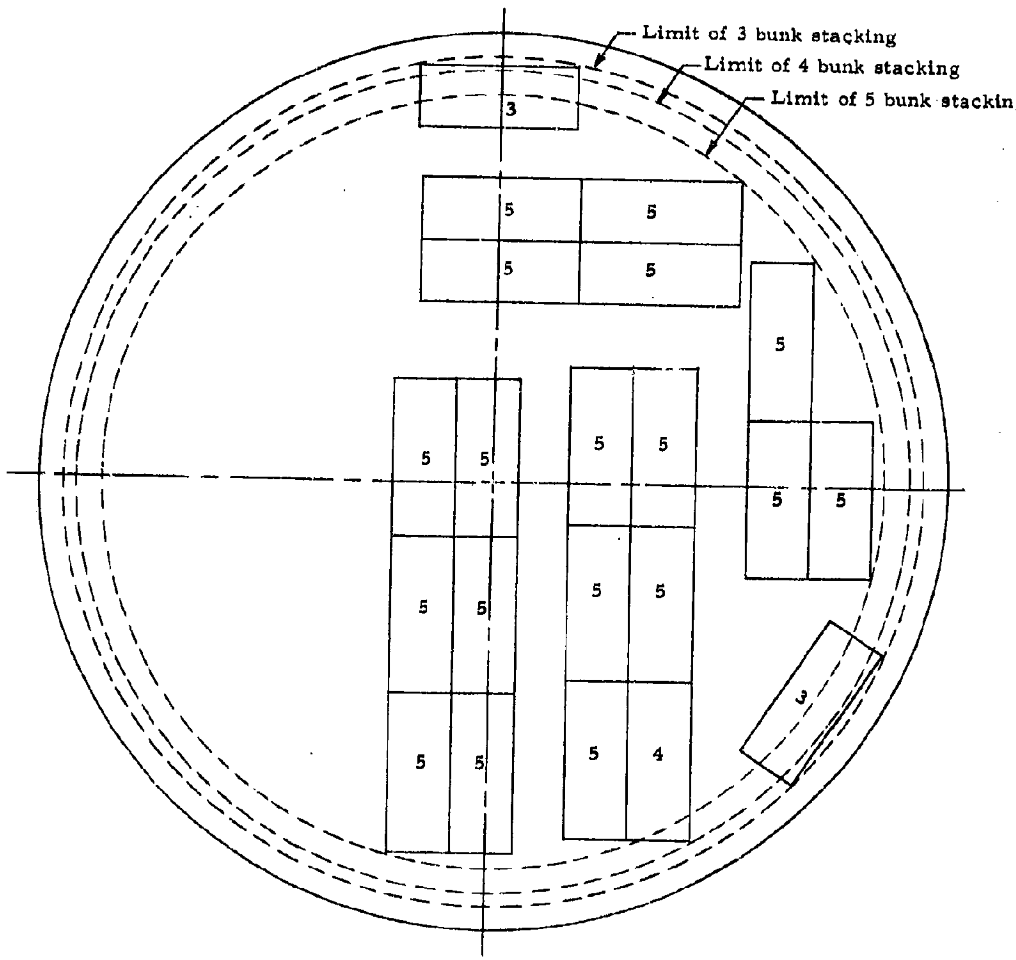


Figure 4-21
FLOOR PLAN OF DOME, 36'-0" DIAMETER
TRIAL DESIGN 4.54B

(b) Steel Shell Design

From Table 3-8 for double curvature plate

Allowable load on 1/4 in. plate f_{dy} = 44,000 psi

$$22{,}000 \text{ lb per in.} > 10{,}800 \text{ lb per in.} \therefore \text{ O.K.}$$

(c) Steel Shell Cost Factors

From Section 2.22

$$X_s = 4.45 \text{ \$/sq ft of shell}$$

(d) Foundation Design

(1) Floor slab design and cost factors

Same as shown in Design Examples 4.34B and 4.34C.

$$C_t = 0.91 \text{ \$/sq ft}$$

(2) Footing design

Load per ft of footing length

$$P = 10{,}800 \times 12 = 129{,}600 \text{ lb/ft}$$

Assuming a soil having an angle of internal friction $\phi = 15^\circ$ and a cohesion factor of c = 2000 psi from Table 3-59 for q = 100 psi, a footing B = 8.0 ft could carry 128 kips per ft. This value is close enough to 129.6 kips per ft actual load to use 8.0 ft wide footing.

$$\frac{P}{L} = \frac{129.6}{8.0} = 16.2 \text{ kips/ft}$$

Assume foundation wall D = 6.00 in.

$$\frac{P}{L} = 16{,}200 \text{ lb}$$

$$\frac{D_{wall}}{L} = \frac{6.00}{8.00} = 0.75$$

Interpolating for f'_c = 6000 psi in Table 3-60

$$\frac{d}{L} = 1.70 \qquad d_{footing} = 1.70 \times 8.0 = 13.60$$

	+ 2.00	cover
Total	15.60 in.	

L = 8.0 ft

Use D = 16.00 in.

Determination of footing main steel ϕ_c

For use with Table 3-61 for d = 13.6 in., L = 8.0 ft, D_{wall} = 6.00 in., P/L = 16.2 kips/ft and f_{dy} = 75,000 psi

$$\frac{d_{footing}}{12\ L - D_{wall}} = \frac{13.60}{96.00 - 6.00} = \frac{13.60}{90.00} = 0.151$$

From Table 3-61

$$\phi_c = 0.91 \text{ percent}$$

(3) Foundation wall design and cost factors

Same as shown in Trial Design 4.44C.

$$C_t = 1.74\ \$/\text{sq ft}$$

(4) Trenching

Depth of trench

$$1.00 + 1.33 = 2.33 \text{ ft}$$

Width of trench

$$8.00 + 1.00 = 9.00 \text{ ft}$$

Cross sectional area of trench

$$2.33 \times 9.00 = 21.0 \text{ sq ft}$$

(e) Foundation Cost Factors

(1) Footing

See Trial Design 4.44C(g) for a sample of detailed cost analysis.

Summary

$$C_c = 1.67$$
$$C_s = 0.89$$
$$C_{st} = 0.11$$
$$C_f = \underline{0.75}$$
$$C_t = 3.42\ \$/\text{sq ft}$$

(2) Trenching

Cost of trench = 0.10 \$/sq ft/ft of trench

$$C_t = 21.0 \times 0.10 = 2.10\ \$/\text{ft of trench}$$

(f) Required Excavation

(1) Minimum depth requirement $h = 3.5$ ft

(2) Depth requirement for full burial

$$h = 0.125\ S_L = 0.125 \times 36.0 = 4.50 \text{ ft}$$

(3) Radiation depth requirement

$$d = (3.5 + 0.02q) = 3.5 + 2.00 = 5.50 \text{ ft}$$

Assume no attenuation through shell wall.

Total depth of excavation

$$z = 0.25 + \frac{36.00}{2} + 5.50 = 23.75 \text{ ft}$$

Total volume of excavation

Determine the volume of the cone with a 1:1 side slope and a base diameter equal to $\left(S_L + 2\ z\right)$ and an altitude equal to $z + \left(S_L\right)$. From this volume subtract the volume of a cone with a base diameter equal to S_L and altitude equal to $S_L/2$.

$$\text{Volume} = \left[\frac{1}{3} \times \pi \times \left(\frac{83.50}{2}\right)^2 \times (23.75 + 36.00)\right] - \left[\frac{1}{3} \times \pi \times \left(\frac{36.00}{2}\right)^2 \times 18.00\right]$$

$$\text{Volume} = 109,000 - 6100 = 102,900 \text{ cu ft}$$

Gross volume of structure

$$\frac{2}{3} \times \pi \times \left(\frac{36.25}{2}\right)^3 = 12,500 \text{ cu ft}$$

(g) Entrance Way

From Table 4-4

$$C_T = \$5620$$

(h) Total Cost

Steel Dome Shell	$4.45 \times 4 \times \pi \times \left(\frac{36.00}{2}\right)^2 \times \frac{1}{2} =$	9050
Floor Slab	$0.91 \times \pi \times \left(\frac{36.00}{2}\right)^2 =$	926
Foundation Wall	$1.74 \times \pi \times 36.0 \times 1.0 =$	197
Footing	$3.42 \times 8.0 \times \pi \times 36.0 =$	3095
Trench	$2.10 \times \pi \times 36.0 =$	238
Excavation	$0.036 \times 102,900 =$	3710
Back Fill	$0.033 \times (102,900 - 12,500) =$	2980
Haul	$0.026 \times 12,500 =$	325
Entrance Way		5620
	Total	\$26,141

See Figures 4-22 through 4-24 for layout of 34.0 diameter two-story dome.

(i) Net Floor Area

Net floor area (headroom $\geq$ 5'-7') is approximately 908 sq ft.

(j) Net Volume

Net volume of shelter is approximately 12,500 cu ft.

4.6 Shelter Entrance Way

4.61 Introduction

When occupancy requirements are limited to a 100 man capacity shelter, one type of entrance way structure is deemed adequate for all shelter considered over the entire loading range. The entrance way is designed to resist the same overpressure as the shelter it serves. Furthermore, the entrance way structure is expected to serve both as an entrance and exit to the shelter.

It is quite possible in the higher overpressure regions that economy might be better served by providing separate entrance and exit structures. In this manner relatively inexpensive non-blast resistance entrance ways with high traffic rates would be supplemented by inexpensive blast resistant exit ways with low traffic capabilities. An analysis of this dual system, however, is beyond the scope of this study.

4.62 Design Assumptions

Based on a comprehensive study of shelter entrance ways [9, 51] a monolithic reinforced concrete cubicle with an interior horizontal clear span of 4.00 ft and an interior vertical clear span of 7.33 ft act as fixed end one-way slabs supporting a uniform lateral load equal to one-half the design overpressure load acting on the shelter. The roof and ground slabs are designed as moment resisting walls supporting a one-way slab. The entrance way is of variable length depending on the depth of burial of the shelter it serves. See Figure 4-25 for entrance way layout.

4.63 Entrance Way Costs

Table 4-4 presents a resume of the pertinent cost parameters associated with entrance ways for a variety of shelter configurations and static overpressures.

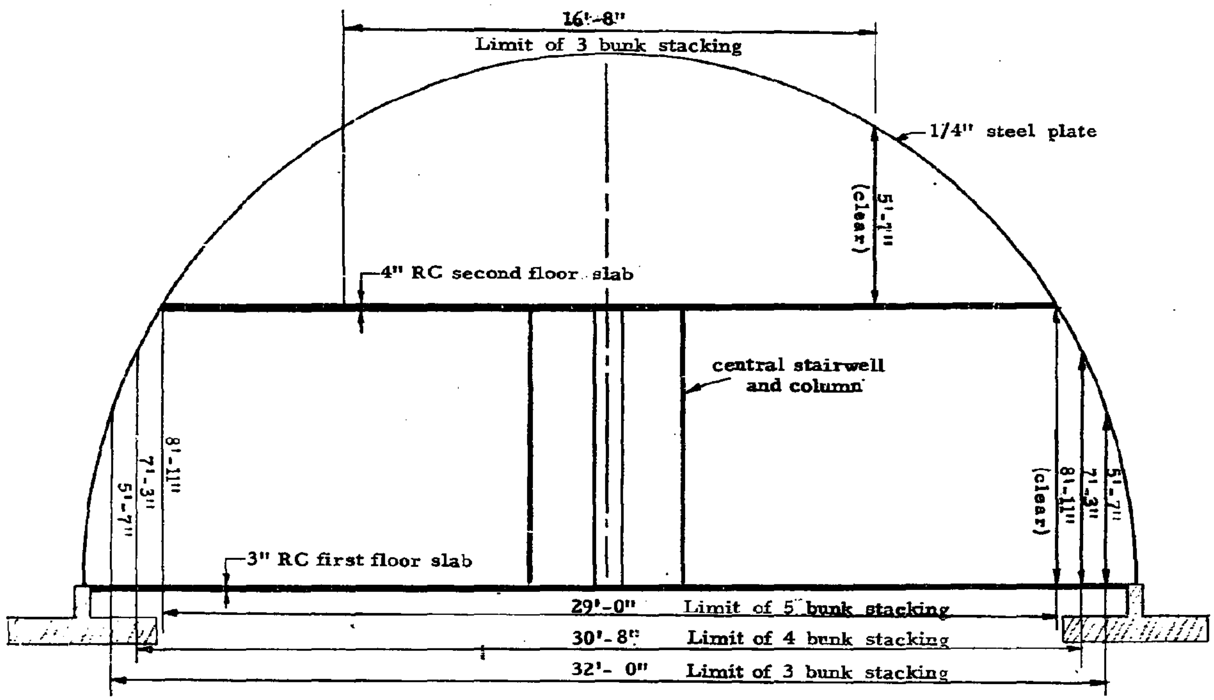


Figure 4-22

TWO STORY DOME, 34'-0" DIAMETER

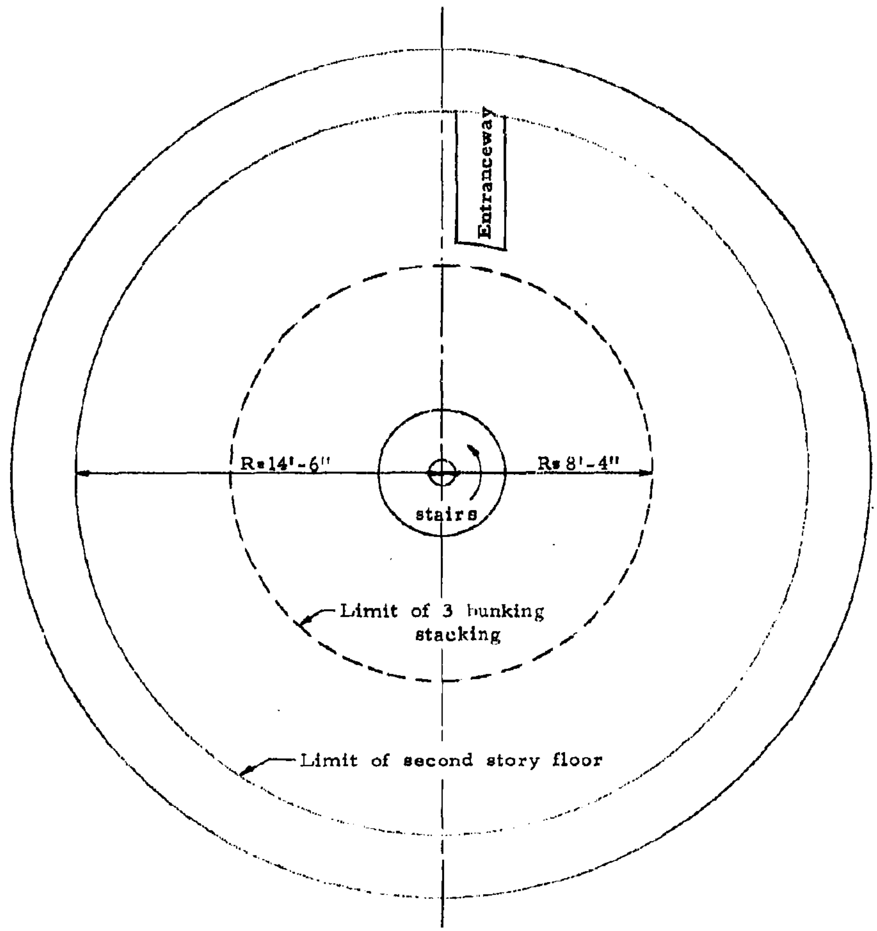


Figure 4-23
SECOND FLOOR OF DOME, 34'-0" DIAMETER

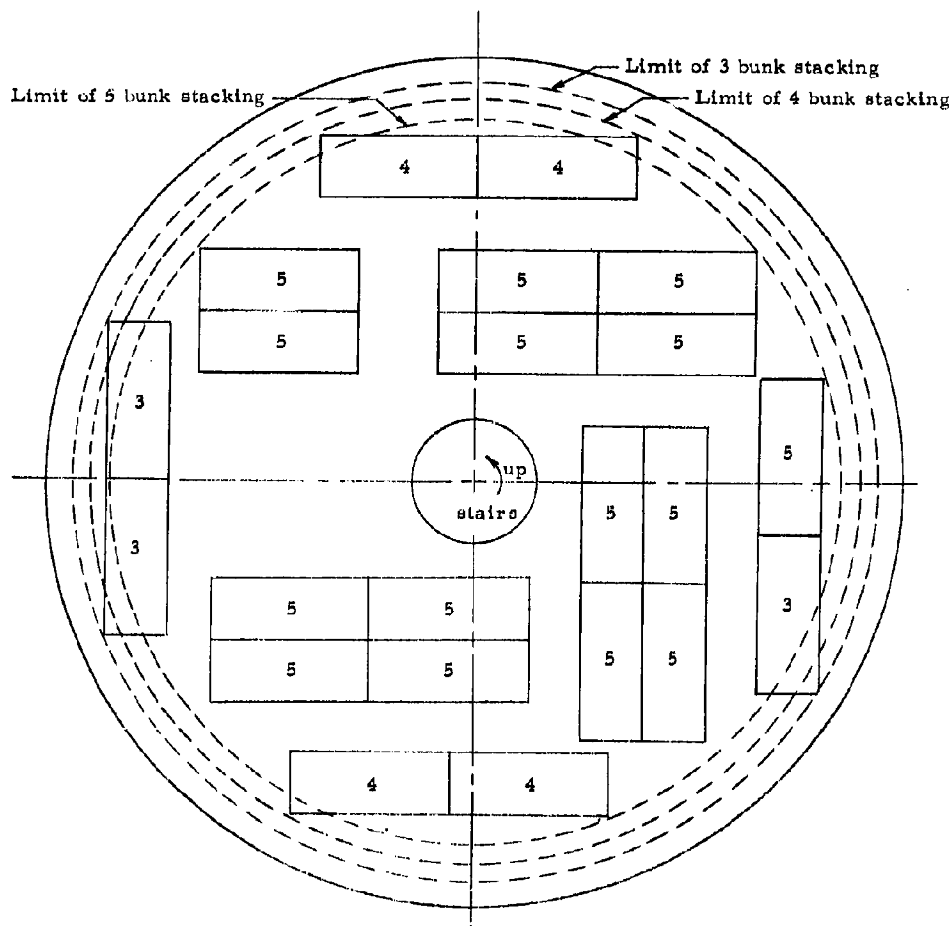


Figure 4-24
FIRST FLOOR OF DOME, 34'-0" DIAMETER

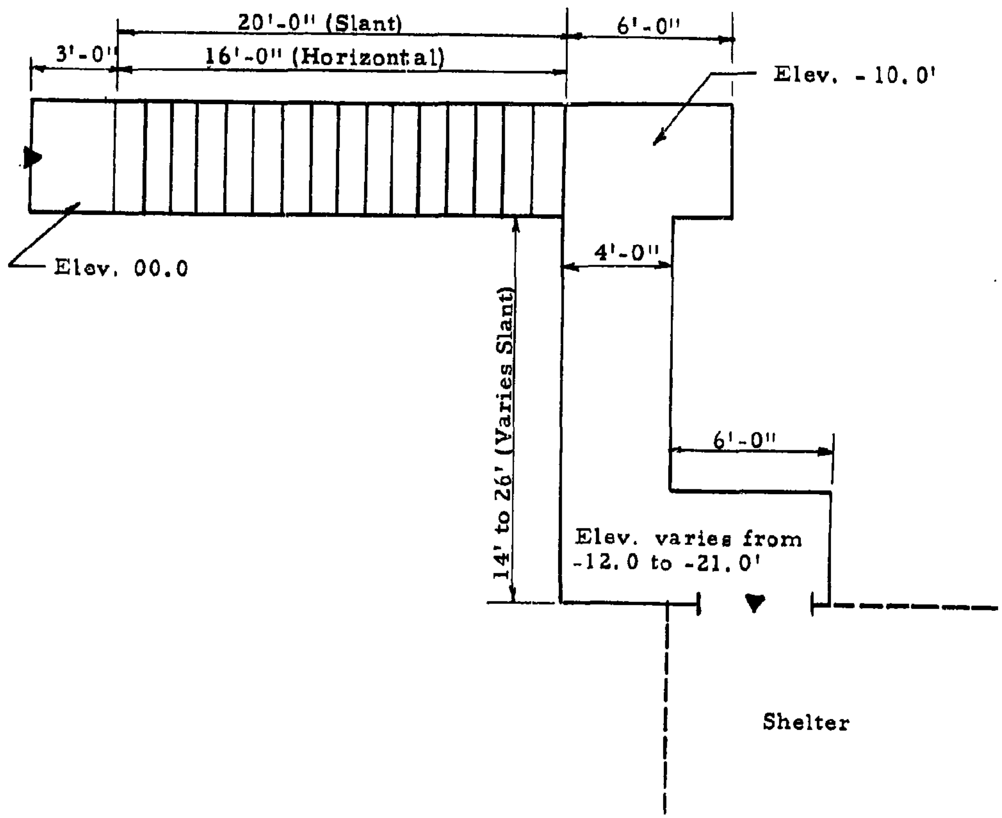


Figure 4-25
PLAN VIEW SHELTER ENTRANCE WAY

CHAPTER 5 OPTIMUM SHELTER CONSIDERATIONS

5.1 Introduction

The recommendations contained in this and earlier chapters, if carefully applied, should normally lead to economic structural designs for a buried 100-man capacity shelter. At the very least, this information will form the basis for a rational preliminary design. Several economic trends in the use of structural materials, which became apparent in the course of this study, will be briefly discussed in the following sections.

5.2 Materials

In Chapter 2, the cost and availability for the major construction materials are examined in some detail. Generalized design and cost relationships are supplied in Chapter 3, while structural costs are evaluated in Chapter 4 for typical 100-man shelters. The most versatile of these construction materials, considering both over-all economy and range of applicability, is reinforced concrete. Its constituent materials are normally available in all regions of the United States, although a shortage of reinforcing steel might be expected under certain conditions (see Chapter 2). As a consequence of its normal availability, plus the widespread familiarity with its use, a relatively short lead-time would be required between the shelter planning and construction phases.

Timber and structural steel shelter elements also show suitability for use in shelter construction, although their economic advantage is restricted to the lower design-pressure ranges. These materials can be considered for use in regions close to their primary centers of supply.

5.3 Costs

The costs developed for materials, for structural elements, and for the entire shelter structure are based on early 1963 prices in the Chicago Metropolitan Area. The cost of the materials, plus fabrication, transportation and erection, form the basis for estimating on-site material costs. To basic

costs are added an additional 40 percent as an allowance for job overhead, general overhead and profit.

5.4 Shelter Elements

5.41 Axially Loaded Compression Member

Cost studies of all the major building materials show that material strength increases more rapidly than the corresponding material cost. The difference in the rate of increase is quite marked and is not likely to be altered by normal price shifts in the near future.

For the assumptions of loading distribution (see Chapter 1) and for the range of structural dimensions used in this study, buckling of buried compression elements is not a prime consideration. Therefore, the use of the highest available strength in a given material normally leads to the lowest element cost. Only when minimum dimension requirements govern design is this situation altered. With regard to reinforced concrete compression members, minimal use of steel reinforcement is recommended for economical design.

5.42 Axially-Loaded Compression Members Subject to Large Bending Moments

With the exception of reinforced concrete elements, the observations made in Section 5.41 apply equally to eccentrically-loaded compression members. In concrete compression members which are subject to tensile failures, the use of low strength concrete together with high strength steel leads to economy of design.

5.43 Flexural Members

The observations of Section 5.41 also apply to flexural members. However, the member must be able to resist an involved interrelationship of moment, diagonal tension and shear. This is particularly true in reinforced concrete members, where material properties and material combinations may be varied separately to obtain an optimum structural element. It is recommended that the design and cost tables presented in Chapter 3 be used when possible.

5.5 Dynamic Loading Characteristics

The structural elements described in Chapter 3, as well as the structures in the sample design problems of Chapter 4, are designed to withstand "equivalent" uniform static loadings, q psi. This use of "equivalent" loading, while permitting major simplifications in the analytical expressions, should be recognized as an artificial concept. Its derivation recognizes the increased structural resistance of many materials to rapid rates of loading, such as those produced by nuclear detonations, but does not consider the response of the structural element or of the structure to a dynamic application of loading. This latter effect must also be considered if structural shelter costs are to be related to levels of surface overpressure.

This study has assumed (see Chapter 1) that the duration of load application due to a nuclear explosion will be long in comparison with the natural period of the element or structure which supports the load. The classical blast loading is analyzed as a loading which reaches its maximum value within a very brief rise time, and subsequently decays at a much slower rate. This type of loading can be closely approximated as a triangular step loading with zero rise time[1, 3]. Its effect on the structure, as compared with a statically-applied load of the same peak magnitude, will be primarily dependent upon the elasto-plastic characteristics of the loaded structure or structural element.

The ductility ratio[2] is a measure of the amount of plastic deformation which is permitted in the structure. This quantity, designated by the symbol, μ, is the ratio of maximum deflection to the elastic limit or yield deflection. Thus, a specified value of $\mu = 1.0$ implies that the material will not be allowed to yield beyond its elastic range. This assumption was implicit in the analysis of timber elements, Section 3.4, since there was no proven basis for assuming plastic action in timber members. For materials and structural systems where it is reasonable to anticipate plastic yielding of critical sections, the use of some value of μ greater than unity is a logical consequence.

A value of $\mu = 1.3$ is considered[2] to correspond to slight damage of an element or structure, since the permanent yield deflection is only 30 percent of the elastic deflection. This value of the ductility ratio is

recommended in Reference 2 for use where sizeable deflections of the elements cannot be tolerated, as is postulated to be the case for domes and arches. A value of $\mu = 3.0$ implies larger permanent deflections, but still without collapse of the element or structure[2]. Values of $\mu = 10.0$ or more have also been recommended[7], particularly for carefully detailed steel elements.

Figure 5-1, taken from Figure 5D-5 of Reference 2, illustrates the relationship between peak dynamic load, p_m, and equivalent static load, q, for an initial-peak, triangular force pulse acting on an elasto-plastic system. The ordinate is the ductility factor, $\mu = x_m/x_y$, while the abscissa is the ratio of load duration to effective natural period of the structure, t_d/T. Finite values of the ratios of peak dynamic force to required yield point resistance, p_m/q, are plotted as continuous curves. Also plotted is the ratio of the time at which maximum deflection is reached to the effective natural period of the structure, t_m/T. Values of p_m/q, as obtained from this chart, can be used to convert "equivalent" static loading to dynamic loadings.

The peak side-on value of the overpressure at the ground surface, p_{so} (psi) must be given or assumed at the onset of design. For shallow buried structures, such as are considered in this study, any attenuation of this peak overpressure due to its passage through the soil will probably be minor. Hence, in general, $p_{so} = p_m$ for horizontal buried surfaces. The combined resistance of the soil-structure system is customarily not examined, primarily due to our lack of understanding of the load-soil-structure interactions. There are, however, many indications that the combined soil-structure strength may differ appreciably from the strength of the structure alone (see Appendix A). The peak horizontal pressure on vertical buried surfaces is taken as some fraction, k_h, of the vertical pressure, whose value is dependent on the soil type.[2, 3]

Cohesionless soil, damp or dry	$k_h = 0.250$
Unsaturated cohesive soil, stiff consistency	$k_h = 0.333$
Unsaturated cohesive soil, medium consistency	$k_h = 0.500$
Unsaturated cohesive soil, soft consistency	$k_h = 0.750$
All saturated soils, water level at surface	$k_h = 1.000$

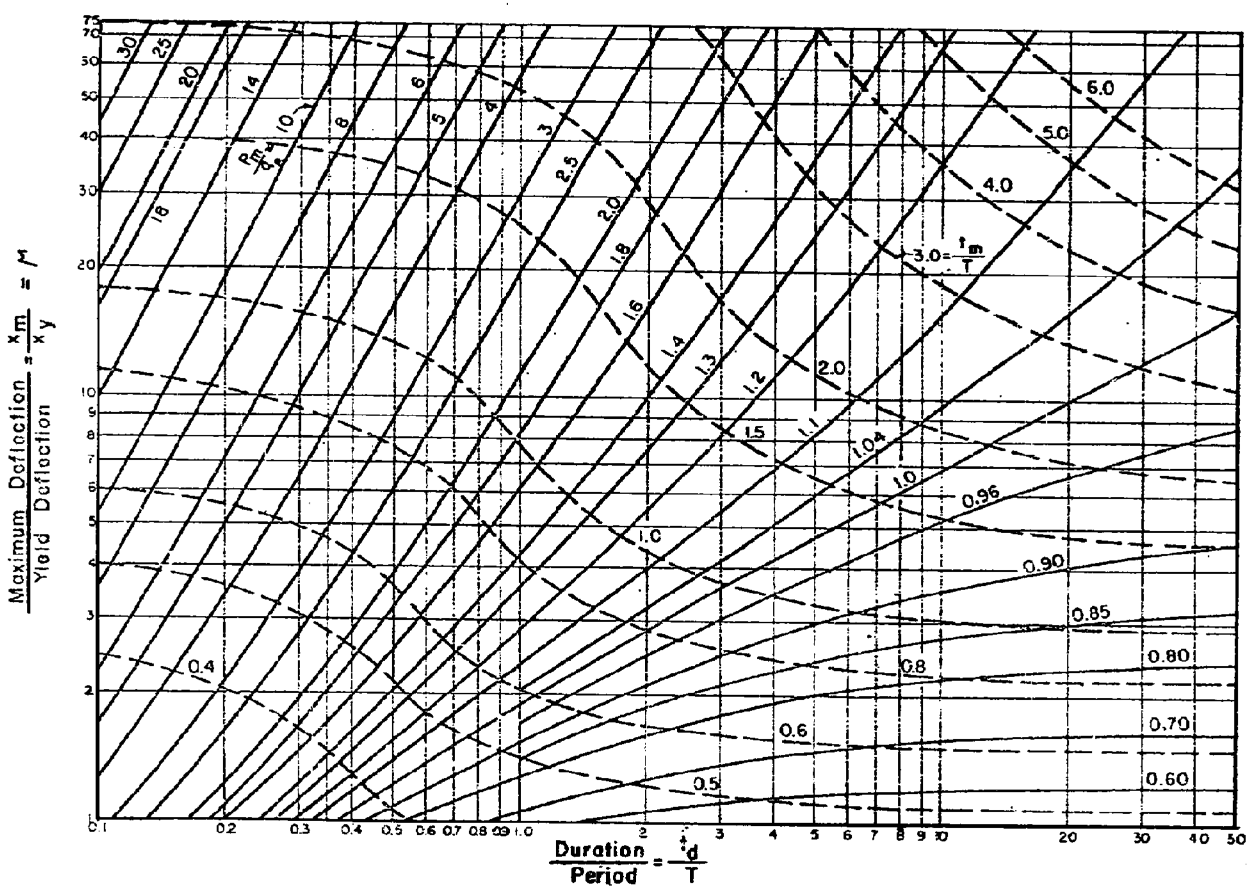


Figure 5-1

EFFECT OF INITIAL-PEAK TRIANGULAR FORCE PULSE
ON AN ELASTO-PLASTIC SYSTEM

Note that the design examples of Chapter 4 have assumed $k_h = 0.500$ for all cases. For many structural elements, such as the eccentrically-loaded side walls of a monolithic cubicle, the choice of k_h has a negligible effect on the structural requirements. However, the design of end walls is influenced by k_h, Shell structures, either doubly or singly curved, are designed for a radial loading of p_m without any consideration of k_h.

With a value thus established for p_m, an assumption must be made as to a permissible value for the ductility ratio, μ, for the element or composite structure. This selection will be influenced by the material properties, the assured continuity of a structure, and the probable consequences of large yielding into the plastic range. (For example, a shelter which will be located below the ground water level probably cannot tolerate any plastic yielding.) Finally, if the ratio of t_d/T is assumed to be large, inspection of Figure 5-1 will confirm that a value of p_m/q corresponding to the assumed value of μ can be estimated with fairly good accuracy. With this accomplished, the required yield resistance (derived for each structural element as resistance to "equivalent" static loading) can be directly related to the design level of overpressure. Approximate values for this relationship, with t_d/T assumed large, are as follows:

Table 5-1

APPROXIMATE RELATIONSHIP BETWEEN PEAK DYNAMIC LOADING ON BURIED STRUCTURE, DUCTILITY RATIO, AND EQUIVALENT STATIC LOADING (Long-Duration Loading Assumed)

Ductility Ratio, μ	Required Value of Equivalent Load, q, (psi)
1.0	2.0 p_m
1.3	1.6 p_m
3.0	1.2 p_m
10.0	1.0 p_m

5.6 Optimum 100 - Man Shelter Structure

As indicated by the trial designs presented in Chapter 4, a large number of possible shelter layouts and configurations exist. A summary of the comparative costs of a number of possible shelter designs, evaluated

over a wide range of overpressure, is presented in Figure 5-2. While all the possible configurations have by no means been examined, Figure 5-2 provides a useful comparison between the various classes of structures.

As can be seen from Figure 5-2, no one structure or configuration is optimum over the entire pressure range. In general, the one-story cubicle is optimum up to 140 psi equivalent static pressure. A variety of material combinations and construction techniques can be utilized for this configuration without serious cost penalty, particularly for the lower equivalent static pressures. It should be noted that, at the lowest loading level, the timber frame cubicle is the most economical type of construction. This design rapidly gives way to the monolithic concrete cubicle, as the design level of loading is increased. While it is not apparent from the figure, concrete and steel frames have a distinct economic advantage when used with masonry block as exterior walls of monolithic structures up to loadings of 25 psi equivalent static pressure. At equivalent static pressures greater than 140 psi, as indicated by Figure 5-2, the 15-ft diameter cylinder of reinforced concrete replaces the cubicle as the least-cost structure.

Figure 5-3 illustrates the optimum structure cost as a function of overpressure, introducing the dynamic loading criteria presented in Section 5.5. Three different types of shelter, all having usable floor areas of approximately 840 sq ft, make up the optimum cost curve. The term "usable floor area" is defined as the interior floor area of the shelter having head room of at least 5.7 feet.

5.7 Blast-Resistant Features in Conventional Construction

This study has indicated that fully-buried culvert and tunnel sections, fabricated from standard gages of corrugated steel plates, can be expected to resist overpressures of 40 to 60 psi when supplementary provisions are made for end closures. Similarly, fully-buried reinforced-concrete cubicles with spans of less than 15-ft and with properly designed and detailed slab roofs of 6 to 8 in. thickness should be able to resist overpressures of approximately 10 psi. These findings suggest the possibility of incorporating at a moderate additional cost, an appreciable measure of blast-resistance into selected portions of new structures. If such a procedure is to be followed, however, the architectural layouts and structural detailing for the protected

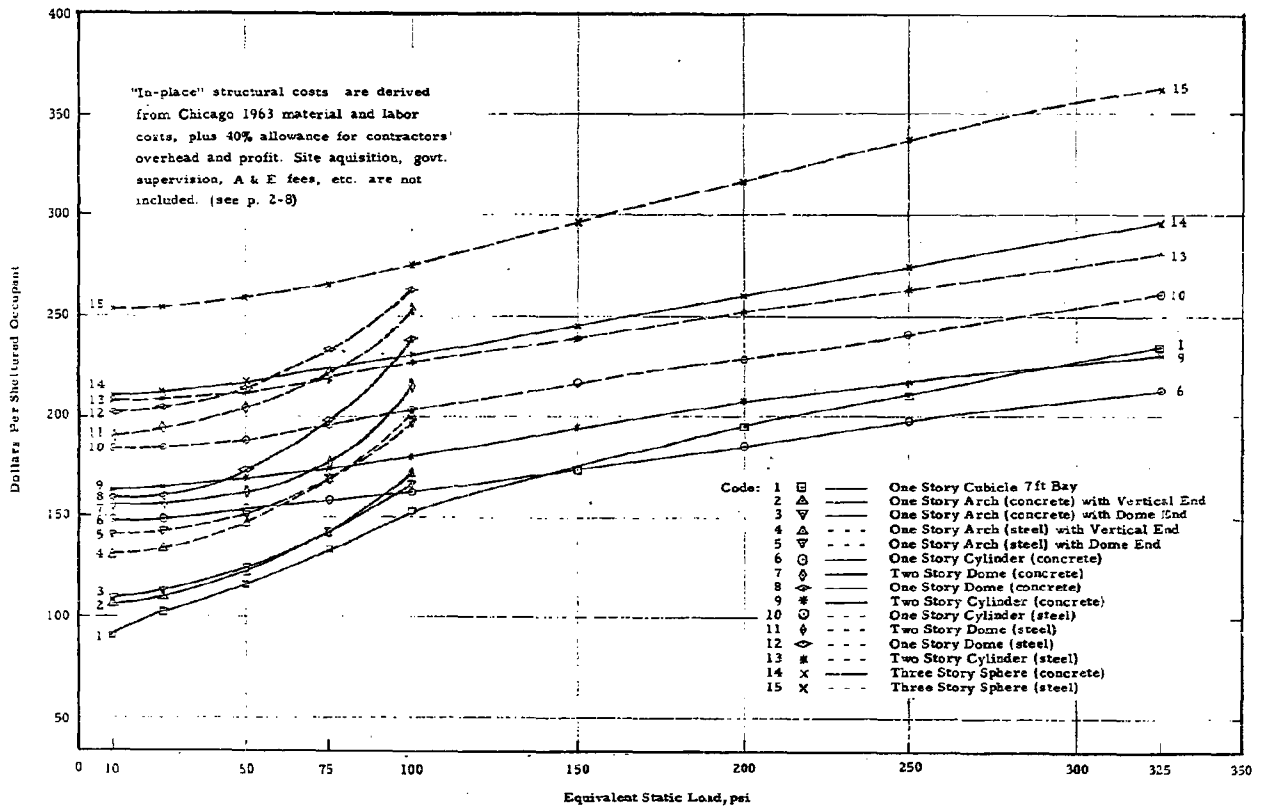


Figure 5-2

IN-PLACE STRUCTURAL COST FOR HARDENED 100-MAN SHELTERS
(Includes Entrance Ways and Excavation)

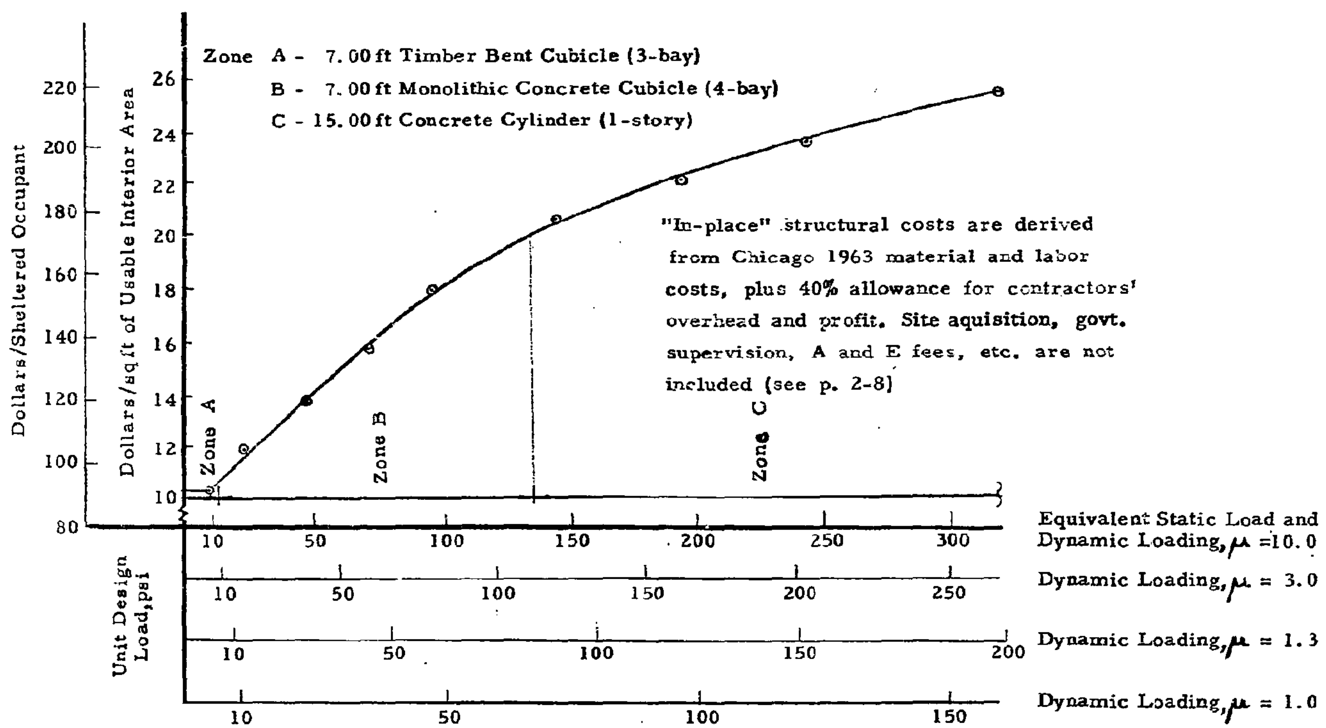


Figure 5-3

MINIMUM IN-PLACE STRUCTURAL COSTS FOR HARDENED 100-MAN CAPACITY SHELTER
(Includes Entrance Ways and Excavation)

area must be based upon a sound appreciation of blast-resistant design. Furtherance of this concept, froquently referred to as "slanted" construction, appears to offer a feasible scheme whereby more than a taken measure of blast resistance can ultimately be afforded the civilian population. Extreme caution should be used in attempting to evaluate the blast-resistant capabilities of existing buildings, however, since seemingly-minor details of reinforcement placement and structural continuity may seriously reduce the blast-tolerance of a conventionally-designed structure.

The ultimate purpose of any personnel protective shelter, regardless of whether it is designed as a separate structure or is incorporated into a conventionally-designed building, is to protect its occupants during some postulated range of nuclear attack environments. In order to supply such protection, the shelter must be designed to minimize the possibility of its structural collapse under the anticipated loadings. Preservation of the structural integrity of a shelter during such hypothesized attacks, however, is not synonymous with the survival of its occupants. Structural survival is almost certainly necessary, but in itself is not sufficient to ensure human survival. Each category of proposed shelter must be designed and analyzed as a balanced protective system, in which adequate structural resistance is only one of several essential components.

BIBLIOGRAPHY

1. Merritt, J. L. and Newmark, N. M., "Design of Underground Structures to Resist Nuclear Blast," Vol. 2, Final Report, Contract No. DA-49-129 eng. 312, University of Illinois for Office of Chief of Engineers, U. S. Army, Washington, D. C., April 1958.

2. Newmark, Hansen and Associates, "Protective Construction Review Guide," Vol. 1, Contract No. SD-52, prepared for the Office of the Assistant Secretary of Defense, Washington, D. C., June 1961.

3. _______, "Design of Structures to Resist Nuclear Weapons Effects," ASCE Manual of Engineering Practice No. 42, New York, 1961.

4. Glasstone, S. (editor), "The Effects of Nuclear Weapons," prepared by the U. S. Department of Defense, published by the U. S. Atomic Energy Commission, Superintendent of Documents, Washington, D. C., April 1962.

5. Newmark, N. M. and Haltiwanger, J. D., "Principles and Practices for Design of Hardened Structures," Contract No. AF 29(601)-2390, Project No. 1080, Task No. 10802, University of Illinois for Research Directorate, Air Force Special Weapons Center, Kirtland A. F. B., New Mexico, December 1962.

6. _______, "Design of Structures to Resist the Effects of Atomic Weapons," Corps of Engineers, E. M. 1110-345-413 through 421, Superintendent of Documents, Washington, D. C.

7. Norris, C. H., Hansen, R. J., et al, "Structural Design for Dynamic Loads," McGraw-Hill Book Company, Inc., 1959.

8. Sievers, R. H., "Protective Construction by Proved Components, Technical Report No. 1689-TR, U. S. Army Engineer Research and Development Laboratories, Corps of Engineers.

9. Krupka, R. A., "Shelter Configuration Factors," Contract No. OCD-OS-62-108, Sub-task 1152 B, prepared for the Department of Defense, Office of Civil Defense, by Guy B. Panero, Inc., New York, April 1963.

10. Forrestal, M. J., "Protection Against High Blast Overpressure and Ground Shock," Contract No. OCD-OS-62-59, Project No. MR 1188, prepared for the Department of Defense, Office of Civil Defense, by the MRD Division of General American Transportation Company, Niles, Illinois, 1962.

11. Manjoine, M. J., "Influence of Rate of Strain and Temperature on Yield Stresses of Mild Steel," Journal of Applied Mechanics, Vol. 11, No. 4, December 1944.

12. Fry, L. H., "Speed in Tension Testing and its Influence of Yield Point Values," ASTM Proceedings, Vol. 40, 1940.

13. _______, "High Strength Steel Concrete Reinforcing Bars," American Iron and Steel Institute, New York, 1961.

14. Watstein, D., "Properties of Concrete at High Rates of Loading," Symposium on Impact Testing, ASTM Special Technical Publication No. 176, 1938.

15. McDowell, E. L., McKee, K. E. and Sevin, E., "Arching Action Theory of Masonry Walls," ASCE Proceedings, Paper No. 915, March 1956.

16. McKee, K. E. and Sevin, E., "Design of Masonry Walls for Blast Loading," ASCE Proceedings, Separate No. 1512, January 1958.

17. _______, "Wood Structural Design Data," 3rd edition, National Lumber Manufacturers Association, Washington, D. C., 1957.

18. _______, "Wood Stresses for Stress Grade Lumber," National Lumber Manufacturers Association, 1962.

19. _______, "Timber Construction Standards," 3rd edition, AITC 100-62, American Institute of Timber Construction, Washington, D. C., 1962.

20. _______, "Wood Handbook, " Handbook No. 72, Forest Products Laboratory, U. S. Dept. of Agriculture, 1955.

21. Peurifoy, R. L., "Estimating Construction Costs," 2nd edition, McGraw-Hill Book Co., Inc., 1958.

22. Pulver, H. E., "Construction Estimates and Costs," 3rd edition, McGraw-Hill Book Co., Inc., 1960.

23. _______, "Engineering News-Record," (current quarterly summaries of construction costs), McGraw-Hill Book Co., Inc., 1962-1963.

24. Godfrey, R. S. (editor), "Building Construction Cost Data, 1962," R. S. Means Co., Duxbury, Mass., 1962.

25. _______, "Annual Statistical Report," American Iron and Steel Institute, New York, 1961.

26. _______, "Directory of Iron and Steel Works of the United States and Canada," Vols. 27, 28 and 29, American Iron and Steel Institute, New York.

27. _______, "Metal Statistics 1962," 55th edition, American Metal Market, New York, 1962.

28. _______, "Mineral Year Book 1955," through "Mineral Year Book 1961," Vol. I, Bureau of Mines, U. S. Department of Interior, U. S. Government Printing Office, Washington.

29. _______, "Statistical Abstract of the United States 1962," Bureau of the Census, U. S. Department of Commerce, U. S. Government Printing Office, Washington, 1962.

30. _______, "Timber Resources for America's Future," Forest Resource Report No. 14, Forest Service, U. S. Department of Agriculture, U. S. Government Printing Office, Washington, 1958.

31. _______, "Commentary on Plastic Design in Steel," ASCE Manual of Engineering Practice No. 41, New York, 1961.

32. Galambos, T. V. and Ketter, R. L., "Columns Under Combined Bending and Thrust," ASCE Proceedings, Paper No. 1990, April 1959.

33. _______, "Commentary on the Specification for the Design, Fabrication and Erection of Structural Steel for Buildings," AISC, November 1961.

34. _______, "Steel Construction," AISC.

35. _______, "Plastic Design in Steel," AISC, January 1959.

36. _______, "Handbook of Drainage and Construction Products," Armco Drainage and Metal Products, Inc., Middletown, Ohio, 1958.

37. _______, "Building Code Requirements for Reinforced Concrete," ACI 318-63.

38. Dunham, C. W., "The Theory and Practice of Reinforced Concrete," 3rd edition, McGraw-Hill Book Co., Inc., 1953.

39. Wood, R. H., "Plastic and Elastic Design of Slabs and Plates," Ronald Press Co., 1961.

40. Mattock, A. H., Kriz, L. B. and Hognestad, E., "Rectangular Concrete Stress Distribution in Ultimate Strength Design," Proceedings of the American Concrete Institute, Vol. 57.

41. Ferguson, P. M., "Reinforced Concrete Fundamentals," John Wiley and Sons, Inc., 1958.

42. Triandafilidis, G. E., "Analytical Study of Dynamic Bearing Capacity of Foundations," Contract No. DA-22-079-eng.-240, University of Illinois for the Defense Atomic Support Agency, January 1961.

43. McKee, K. E. and Shenkman, S., "Design and Analysis of Foundation for Protective Structures, "Final Report, Project 1080, Task No. 10803, Armour Research Foundation for Research Directorate, Air Force Special Weapons Center, Kirtland AFB, New Mexico, January 1962.

44. Shenkman, S., "A Study of the Design and Analysis of Foundations," Contract No. AF29(601)-5197, Armour Research Foundation for Air Force Special Weapons Center, Kirtland AFB, New Mexico, June 1963.

45. _______, "Shear and Diagonal Tension," Report of the Joint ASCE-ACI Committee on Shear and Diagonal Tension, Proceedings of the American Concrete Institute, Vol. 59, January-March, 1962.

46. Moe, J., "Shearing Strength of Reinforced Concrete Slabs and Footings Under Concentrated Loads," Bulletin D 47, Portland Cement Association, April 1961.

47. Massachusetts Institute of Technology, "Behavior of Wall Panels Under Static and Dynamic Loads II," Department of Civil and Sanitary Engineering, January 1954.

48. Curoine, C, "Cost Guide for Protective Construction," Budocks Technical Digest No. 91, November-December 1952.

49. Glasstone, S. (editor), "The Effects of Nuclear Weapons," prepared by the U. S. Department of Defense, published by the U. S. Atomic Energy Commission, Superintendent of Documents, Washington, D. C., June 1957.

50. American Concrete Institute, "Reinforced Concrete Design Handbook," Report by Committee 317, American Concrete Institute, 1955.

51. Barnett, R. L., "Design of Entrance Systems for Personnel Protection Shelters," Final Report, Contract No. NBy-3163, Armour Research Foundation for U. S. Naval Civil Engineering Laboratory, Port Hueneme, California, December 1959. (Secret)

APPENDIX A

EXPERIMENTAL STUDY OF THE RESPONSE OF BURIED STRUCTURAL ELEMENTS TO STATIC AND DYNAMIC SURFACE LOADING

by

J. Havers and W. Truesdale

APPENDIX A

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EXPERIMENTAL STUDY OF THE RESPONSE OF BURIED STRUCTURAL ELEMENTS TO STATIC AND DYNAMIC SURFACE LOADING

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INTRODUCTION

When a proposed structure must be "hardened" to withstand the direct effects of a thermonuclear explosion, preliminary projections of construction costs frequently show that it will be advantageous to place the entire structure below finished grade. This may be true even when relatively low overpressures are specified in the design criteria, since a shallow burial-depth both provides a high degree of radiation protection and essentially eliminates any reflection of pressure at the surface interface. Further, based on empirical conclusions drawn from field tests, it is frequently assumed(1, 2)* that the lateral earth support resulting from shallow burial is sufficient to inhibit the primary buckling modes in many arched and domed structures. As a consequence, such structures can be expected to develop greater ultimate loading resistances than in comparable above-ground structures.

Apart from these features, however, conventional design procedures give little recognition to other possible benefits resulting from soil-structure interaction. It has been postulated, based on analytical studies and field observations, that the free-field earth pressure at the level of a shallow buried structure is only slightly less than the surface side-on overpressure. As a consequence, such structures are frequently designed to withstand a dynamic loading which is directly related, through blast wave and structural parameters, to the full surface overpressure(3).

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*Numbers in parentheses refer to references listed in the bibliography included in this paper.

The effects of a surface-applied load on a buried structure must be transmitted through the soil cover. Consequent deformation of the structure must, if loading is to be maintained on the structure, be accompanied by deformation of the cover soil. If the soil resists this deformation, as evidenced by the development of shearing stresses within the soil mass, the soil strength thus mobilized by the cover soil in resistance to its downward movement must, for any conservative system of displacements, complement the resistance furnished by the structure itself.

An effective increase in structural resistance as a result of stress redistribution within the mass of cover soil has, to a limited extent, been recognized in proposed design procedures(1, 2, 4). The absolute contribution which soil strength may contribute to the total soil structure resistance, as well as the relative influence of the several surface load-structure-soil parameters, is almost totally unknown. This lack of knowledge is a matter of concern since there is an excellent possibility that, through a better understanding of soil-structure interaction, major economies can be realized in the design of hardened structures.

A theory to predict the effect of loading on a buried structure, accompanied by a limited series of static loading tests on buried rigid and flexible panels, has been described by other authors(5). This theory predicted that the effect of a dynamic surface loading on a buried structure would be significantly reduced by inertial forces and by internal sliding resistances within adjacent soil masses prior to failure. Experimental results for the limiting condition of static loading confirmed that the reduction in structural loading was significant, within the limits of the test program. By introducing appropriate soil strength values, reasonable correlation was obtained between test results and theory.

In this earlier laboratory investigation, rigid and flexible roof panels were buried at successively-increasing depths in a dense dry Ottawa sand, and subjected to a statically-applied uniform surface load. The panels were 4-in. x 4-in. in size, fabricated from 0.01-in. shim steel stock. The tests were performed in a "glass-box" apparatus(6), thus permitting visual observations of soil and panel deformations under increasing load. The test conditions were considered to approximate two-dimensional tests

since preliminary calibration studies indicated that any frictional forces developed between the sand and the glass side-walls should be relatively small. The observed data consisted of applied surface load and the corresponding central deflection of the panel. Ultimate loads, for both the rigid and flexible panels, were usually identified by an accelerated collapse of the yielding panel. The magnitude of surface loading which was necessary to cause this collapse increased rapidly with depth of burial, as had been postulated by the theory. At a depth of burial of 1-1/2 panel widths, neither type of panel could be collapsed within the 20 psi load limit of the equipment. The central deflection of the panels had then reached approximately 10 per cent of the panel span.

A study of the cost of buried structural elements[7] utilized this theory to predict the actual loading on buried structures. Its finding indicated that the depth of burial associated with minimum structural costs is frequently in excess of the depths required to satisfy "full burial" criteria or to furnish satisfactory radiation protection. The possible savings in the cost of buried structures, as suggested in that study, were of major significance. Thus, when initiating a investigation of materials for use in underground structures, sponsored by the Office of Civil Defense, it was decided to extend the earlier experimental work by examining other soil and loading conditions.

EXPERIMENTAL PROGRAM

The current test program was restricted to flexible panels buried at varying depths in sand and in clay soils. Two-dimensional tests were generally employed, very similar to those used in the earlier study. The equipment was strengthened, however, which permitted tests to be performed at a higher surface pressure and at greater burial depths. In addition, a limited series of three-dimensional tests was conducted in a pressure vessel, using an elongated flexible panel and a sand soil. By including these tests, it was possible to examine the influence of the testing environments on the results which were obtained. Finally, by inserting the glass-box apparatus into an air-actuated shock tube, a condition of surface dynamic loading was simulated for comparison with the results obtained in static load tests.

The flexible panel used in the two-dimensional tests consisted of a 4-in. x 4-in. flexible panel made from two sheets of 0.01-in. thickness of shim steel stock. The panel was supported on its two sides by a panel support system mounted on a 3/4-in. diameter rod. This rod passed through a thrust bearing and rested on a force washer, thus permitting the measurement of total load on the panel. (Figure A-1 illustrates the equipment used in the two-dimensional tests. The relatively large members in the support system were found to be necessary to reduce the frequency response of the system under dynamic loading). Central deflections of the panel were optically measured from photographs taken as loading was applied to the surface of the soil.

An elongated flexible panel was used in the three-dimensional tests in order to minimize the relative importance of end effects. As indicated in Figure A-2, a panel with a 4-in. span and a 24-in. length was selected for these tests. The panel was constructed from the same steel shim stock as the panel used in the two-dimensional tests. It was supported along its elongated edges in similar fashion to that described for the two-dimensional tests, except that two 3/4-in. diameter rods were utilized.

The test chamber for the two-dimensional tests consisted of a steel framed box with a height of 24-in., a length of 24-in., and a width of 4-in. (see Figure A-3). The front face of the box consisted of a panel of plate glass, and a plate glass liner was placed against the rear face. As stated earlier, previous experience with similar apparatus had indicated that the friction between a sand and the glass sides was relatively small. As a precaution, however, the glass was sprayed with a clear lubricant when a clay soil was used in the tests. The test chamber used in the three-dimensional static tests was a 36-in. diameter by 36-in. depth pressure vessel, fabricated from steel plate. No special precautions were used to reduce friction between the soil and the chamber walls, since the chamber dimensions were large compared with those of the test specimen.

The surface load in all static tests was applied by means of air pressure, while the shock tube shown in Figure A-4 was used as the loading mechanism for the dynamic tests. This shock apparatus consisted of seven steel hollow-box sections, each 36-1/2-in. in length, which were connected

in series to form a continuous tube. One end-section functioned as the driver, and was sealed from the rest of the tube by means of a plastic membrane. After air pressure in the driving chamber had been increased to a pre-determined level, the membrane was abruptly ruptured. This generated an advancing shock wave, whose typical characteristics are illustrated in Figure A-4. The glass-box apparatus was inserted in the third section from the driver. In this location, a clean shock front would develop upstream from the test chamber and the positive pulse could clear the test specimen before a reflected wave was encountered.

In order to examine the influence of soil strength properties on the loading response of a buried flexible panel, two types of soil were used in the tests. A dry Ottawa sand placed at a medium density, (106 lbs per cubic ft) was used to represent those soils whose shearing strength is dependent upon intergranular sliding resistance. Earlier triaxial tests had indicated that this sand, for conditions similar to those of this test, should develop an effective angle of sliding resistance (ϕ_e) of approximately 33°. A second soil, whose shearing strength was primarily dependent upon cohesion, was prepared by combining 95 per cent by weight of kaolinitic clay with 5 per cent bentonite. The Atterberg Limits for this soil were determined to be: plastic limit of 28 per cent, liquid limit of 68 per cent, plasticity index of 40. The use of this soil, since its composition was controlled, ensured a high degree of uniformity for the several test specimens. The soil moisture content was maintained between 36 and 39 per cent, corresponding to unconfined-compressive strengths of 2.80 to 3.40 psi.

When placing the clay soil for the two-dimensional tests, the glass-box was placed on its back and the glass front removed. The test panel and support assembly, with a spacer block under the panel to prevent initial deflection, were then installed. Clay was hand-tamped in uniform layers, with careful control of its moisture content during placing. Thus, any layering effects due to the method of placement would be averaged over the plance of panel deformation. Sand was placed, in both the two-dimensional and three-dimensional tests, by pouring it from a controlled height.

After the soil had been placed in the glass-box apparatus the glass front panel was removed and 1/2-in. x 1/2-in. grid, delineated with 1/16.-in. width lines of colored sand, was marked on the screeded surface of the compacted soil. Comparisons between this grid and a similar grid on the glass panel permitted a visual observation of deformations in the soil as the panel deflected under applied loading.

TEST RESULTS

The symbols and terms which are used in the presentation of results and the subsequent discussions are now defined:

W	=	applied surface loading (lb), calculated as the product of applied surface pressure and plan area of the panel. When considering dynamic loading, the applied surface pressure is taken as the measured peak dynamic pressure.
w	=	observed load in the panel, (lb)
B	=	span length of flexible panel, (in.) between supported edges.
D	=	vertical distance, (in.) between ground surface and buried flexible panel, prior to application of surface loading.
Δ	=	measured central deflection flexible panel, (in.) under applied load.
r_b	=	panel burial ratio, computed as the quotient of depth of cover soil over the panel divided by the distance between panel edge supports. Thus, $r_b = \frac{D}{B}$.
r_Δ	=	panel deflection ratio, calculated as the quotient of cumulative central panel deflection divided by the distance between panel edge supports. Thus, $r_\Delta = \frac{\Delta}{B}$.
$W(r_\Delta, 0)$	=	applied surface load corresponding to a specified panel deflection ratio and a zero panel burial ratio.
$W(r_\Delta, r_b)$	=	applied surface load corresponding to specific values of panel deflection ratio and panel burial ratio.

$w(r_\Delta, r_b)$	=	observed load on the panel corresponding to specific values of panel deflection ratio and panel burial ratio.
Equivalent Load	=	the applied surface load $W(r_\Delta, 0)$ which, for panel burial ratio r_b equal to zero, produces a specified value of the panel deflection ratio.
Equivalent-Load Factor	=	This is the quotient of the magnitude of singly-applied surface load which produces a specified panel deflection ratio r_Δ, assuming a panel burial ratio of zero, divided by the actual surface load which produces the same panel deflection ratio at a finite level of panel burial ratio; algebraically, this is $\frac{W(r_\Delta, 0)}{W(r_\Delta, r_b)}$, or since $W(r_\Delta, 0) = w(r_\Delta, 0)$, it can also be expressed as $\frac{w(r_\Delta, 0)}{W(r_\Delta, r_b)}$.
Observed-Load Factor	=	This is the ratio of observed load on the panel to applied surface loading, calculated for specified values of panel deflection ratio r_Δ and panel burial ratio r_b. Algebraically it is equal to $\frac{w(r_\Delta, r_b)}{W(r_\Delta, r_b)}$.
Load Redistribution Index, I_r	=	This is the quotient of measured load on the panel at specified values of panel deflection ratio and panel burial ratio, divided by that magnitude of singly-applied surface load which results in the same panel deflection ratio for a zero level of panel burial ratio. Algebraically, $I_r = \frac{w(r_\Delta, r_b)}{w(r_\Delta, 0)}$.

For each test in those series where the surface loading was statically applied, the loading pressure was increased incrementally until the maximum desired value of panel deflection ratio had been reached. At each loading level, observations were made of the total load carried by the panel and of the

TABLE A-1

SUMMARY OF TWO-DIMENSIONAL, STATIC LOADING TESTS

Soil Type	Burial Ratio (r_b)	Panel Deflection Ratio (r_Δ)	Applied Surface Load (lb)	Observed Load on Structure (lb)	Observed Load Factor	Equivalent Load Factor	Structural Load Re-distribution Ratio (I_r)
Sand	0.5	0.025	20.1	2.6	0.130	0.122	1.06
		0.050	42.5	5.2	0.123	0.115	1.06
		0.075	68.0	8.8	0.130	0.108	1.20
		0.100	96.0	19.2	0.200	0.102	1.96
	1.0	0.025	76.5	6.5	0.085	0.032	2.66
		0.050	106.3	15.8	0.149	0.046	3.23
		0.075	129.0	24.5	0.190	0.057	3.34
		0.100	160.5	32.1	0.200	0.061	3.28
	1.5	0.125	94.2	8.8	0.094	0.026	3.60
		0.050	188.4	17.7	0.094	0.026	3.62
		0.075	282.6	28.4	0.097	0.026	3.87
		0.100	376.8	36.9	0.098	0.026	3.77
	2.0	0.025	79.0	7.7	0.097	0.031	3.15
		0.050	158.0	14.5	0.092	0.031	2.96
		0.075	204.0	17.8	0.087	0.036	2.42
		0.100	326.0	27.7	0.085	0.030	2.83
	2.5	0.025	111.3	11.5	0.102	0.022	4.70
		0.050	204.0	15.7	0.077	0.024	3.21
		0.075	319.0	24.9	0.078	0.023	3.39
		0.100	408.0	31.8	0.078	0.024	3.75

TABLE A-1 (con't)

Soil Type	Burial Ratio (r_b)	Panel Deflection Ratio (r_Δ)	Applied Surface Load (lb)	Observed Load on Structure (lb)	Observed Load Factor	Equivalent Load Factor	Structural Load Re-distribution Ratio (I_r)
	3	0.025	188.5	14.7	0.078	0.013	6.00
		0.050	306.0	18.9	0.062	0.016	3.86
		0.075	432.0	26.3	0.061	0.017	3.58
		0.100	575.0	33.9	0.059	0.017	3.46
	4.5	0.025	163.5	9.0	0.055	0.015	3.68
		0.050	350.5	19.3	0.055	0.014	3.94
		0.075	565.0	31.1	0.055	0.013	4.24
		0.100	753.0	39.9	0.053	0.013	4.07
Clay	0.5	0.025	64.5	20.9	0.325	0.037	8.53
		0.050	92.5	42.3	0.457	0.053	8.64
		0.075	108.0	63.7	0.590	0.068	8.67
		0.100	122.5	82.8	0.680	0.080	8.45
	1.0	0.025	122.5	22.3	0.182	0.020	9.10
		0.050	181.0	45.3	0.250	0.027	9.25
		0.075	193.0	61.9	0.320	0.038	8.43
		0.100	208.0	72.2	0.374	0.047	7.37
	1.5	0.025	98.0	9.2	0.094	0.025	3.76
		0.050	132.0	19.8	0.150	0.037	4.05
		0.075	171.0	31.4	0.184	0.043	4.28
		0.100	196.0	44.1	0.225	0.050	4.50
	2.0	0.025	129.0	16.8	0.130	0.019	6.86
		0.050	188.0	33.8	0.180	0.026	6.90
		0.075	245.0	51.5	0.210	0.030	7.01
		0.100	280.0	64.5	0.230	0.035	6.59

TABLE A-2

SUMMARY OF THREE-DIMENSIONAL, STATIC LOADING TESTS

Soil Type	Burial Ratio (r_b)	Panel Deflection Ratio (r_Δ)	Applied Surface Load (lb)	Observed Load on Structure (lb)	Observed Load Factor	Equivalent Load Factor	Structural Load Re-distribution Ratio (I_r)
Sand	0.5	0.025	102	33	0.320	0.164	1.98
		0.050	250	77	0.308	0.133	2.32
		0.075	405	107	0.263	0.124	2.14
	1.0	0.025	633	96	0.144	0.025	5.76
		0.050	1390	201	0.144	0.024	6.03
		0.075	2160	252	0.116	0.023	5.05
	1.5	0.025	864	99	0.114	0.019	5.95
		0.050	2760	215	0.078	0.012	6.45
		0.075	5860	327	0.056	0.009	6.53
	2.0	0.025	912	106	0.115	0.018	6.35
		0.050	2790	240	0.086	0.012	7.20
		0.075	5520	293	0.053	0.009	5.86

TABLE A-3

STUDY OF TWO-DIMENSIONAL, DYNAMIC LOADING TESTS

Soil Type	Burial Ratio (r_b)	Panel Deflection Ratio (r_Δ)	Applied Surface Load (lb)	Observed Load on Structure (lb)	Observed Load Factor	Equivalent Load Factor	Structural Load Redistribution Ratio (I_r)
Sand	0.5	0.147	165.0	22.0	0.133	0.0875	1.52
	1.0	0.063	171.0	15.0	0.088	0.0364	2.42
	1.5	0.032	163.0	5.2	0.032	0.0190	1.68
	2.0	0.021	178.0	8.1	0.045	0.0110	4.10
Clay	0.5	0.065	166.0	21.0	0.126	0.0382	3.3
	1.0	0.008	171.0	4.2	0.025	0.0047	5.3

TABLE A-4

LOAD-DEFLECTION DATA FOR TEST PANELS
AT SOIL SURFACE

Test Panel	Deflection Ratio (r_b)	Uniform Load (psi)	Total Load (lb)
Two-Dimensional	0.0125	0.068	1.09
	0.0250	0.156	2.50
	0.0375	0.228	3.65
	0.0500	0.308	4.93
	0.0625	0.396	6.34
	0.0750	0.458	7.33
	0.0875	0.538	8.61
	0.1000	0.608	9.74
	0.1125	0.678	10.88
	0.1250	0.758	12.14
Three-Dimensional	0.0125	0.084	8.06
	0.0250	0.174	16.70
	0.0375	0.252	24.20
	0.0500	0.334	32.05
	0.0625	0.432	41.50
	0.0750	0.518	49.75
	0.0875	0.602	57.80
	0.1000	0.688	66.05
	0.1125	0.784	75.25
	0.1250	0.880	84.50

cumulative central deflection of the panel. Surface applied pressure and observed panel load could thus be correlated with increasing stages of panel deflection. In the dynamic test series, since a single loading was dynamically applied to the surface of the soil, only the maximum panel deflection and the peak load on the structure could be measured.

Observed values of surface load W, panel load w, and central panel deflection Δ, are tabulated for reference. For use in subsequent evaluations, these tables include computed values for observed-load factors, equivalent-load factors, and load redistribution indicies.

DISCUSSION OF RESULTS

For specific test conditions, observed values of surface load W and panel load w can immediately be related to each corresponding value of central panel displacement. However, in order to study performance trends, it is necessary to examine the inter-relationships between the several test conditions. The observed-load and equivalent-load factors have been introduced for this purpose. These factors, by their definitions, relate surface load and panel load for specific values of panel burial ratio and panel deflection ratio. Thus, an increase in the equivalent-load factor at a fixed level of r_b, as r_Δ is increased, means that the applied surface load (W) is increasing less rapidly than panel deflection. This, in turn, indicates that the resultant of the half-span loading is moving toward the panel center and away from the supported edge. An equivalent-load factor of unity indicates a uniform distribtuion of panel load.

If the observed-load factor is found to increase with increasing panel deflection ratio, it can be concluded that the buried panel is supporting an increasing share of the applied surface load. This would suggest that the strains in the cover soil are increasing more rapidly then is its shearing resistance.

The calculated values of the load-redistribution index, computed as the quotient of the observed-load factor and the equivalent-load factor, provide useful indications of the probably non-uniform distribution of actual loading on the panel. Similarly, the absolute differences between plots of the two load factors are, to some constant scale, measures of this same non-uniformity of panel load. This is true since:

$$\text{observed-load factor} = \frac{w(r_\Delta, r_b)}{W(r_\Delta, r_b)}$$

$$\text{equivalent-load factor} = \frac{w(r_\Delta, 0)}{W(r_\Delta, r_b)}$$

$$\text{difference} = \frac{w(r_\Delta, r_b) - w(r_\Delta, 0)}{W(r_\Delta, r_b)}$$

Thus, when evaluating plots of load factors for a specific panel deflection ratio, the relative slopes of the two plots becomes of some importance. Divergence of these curves suggests that an increasing portion of the panel load is concentrated near the panel supports. The converse, by similar reasoning, should be equally true.

The buried flexible panels, for all stable loading conditions, must be in equilibrium under vertical forces. Nonlinear relationships must almost certainly exist between magnitude of applied surface load area over which such load is applied, and loading response of a buried structure. An analytical approximation can be obtained by substituting a uniform surface pressure, acting over a finite surface area, for the actual loading condition. A further approximation is an assumption that this idealized loading area remains unaffected by changes in magnitude of surface loading and depth of burial. On this basis, thevolume of cover soil affected by incipient failure of a soil-structure system can be considered to be bounded by the ground surface, the surface of the buried structure, and potential sliding surfaces extending from the periphery of the panel to their intersections with the ground surface. By this assumption, it follows that any difference between applied surface load $W(r_\Delta, 0)$ and measured panel load $w(r_\Delta, r_b)$ is transfered to the soil by mobilizing some portion of the soil shearing strength along the potential sliding surfaces. The analyses of these tests results have assumed that these potential failure surfaces consist of vertical planes, hence, the effective area over which a surface load is applied becomes equal to the area of the test panel.

Neglecting possible inertial effects, and postulating that the difference between applied surface load and observed panel load is transferred to the soil above a buried structure, the expressions for the load factors can be written as:

$$\text{observed-load factor} = \frac{w(r_\Delta, r_b)}{w(r_\Delta, r_b) + w\ \underline{\text{soil}}}$$

and

$$\text{equivalent-load factor} = \frac{w(r_\Delta, 0)}{w(r_\Delta, r_b) + w\ \underline{\text{soil}}}$$

where w soil is the load carried by the cover soil.

It can further be reasoned that finite strains must be developed within the soil body above a buried structure, if soil shearing resistance is to be mobilized. The stress-strain relationship may or may not be linear, depending on the soil, but increasing localized strains will be associated with increasing localized stresses until peak resistances have been reached. While such relationships are valid for localized points within the soil body, immediate difficulties are encountered in extrapolations on a global basis to the entire mass of soil above a buried structure. If the effect of a surface load on a soil structure combination is such that a non-uniform distribution of shearing strains results within the soil body, a non-uniform distribution of interior stresses will also exist. Further, due to the general non-linearity of the problem, the relationship between integrated stresses and varying levels of integrated strains, as referenced to the total depth of cover soil, is largely a statistical one.

For any total shearing strain over the depth of cover soil, the stresses at localized points are related only to localized strains. In consequence, it is conceivable that the peak strengths at certain points within the soil body could be exceeded at loadings much less than would be predicted by a consideration of average strains and stresses across the soil depth. Thus, a localized failure may limit the load-resisting capacity of the cover soil, rather than the possibility of a general failure affecting the entire soil body.

To avoid the complexities introduced by considering stress-strain relationships within a soil body, it is customary to treat the soil as a free body subjected to specified boundary forces. Using this approach, the maximum load-resisting capacity can be associated with some finite total strain between the surface of the soil and its lower boundary. The internal distribution of this strain, as well as the internal variation of the shearing stresses which combine to furnish total load resistance, remain unknown. This approach, in effect, replaces interior stress-strain considerations by a weighted averaging

of the localized stresses and strains throughout the soil mass. While this is a major simplification, it involves some important assumptions as to the boundaries of the failure mass of soil. It is frequently assumed[6] that the entire mass of soil between the ground surface and the buried structure, bounded by vertical planes delineating the periphery of the structure, constitutes the incipient failure mass. However, there remain the possibilities that the actual bounds may be influenced by localized stresses and strains and that failure planes other than those assumed may, in fact, prove critical.

If the stress-strain relationships within a loaded soil mass are such that the total shearing resistance of the soil does not increase as failure becomes imminent, the failure planes developed within the soil should ultimately extend throughout the entire soil depth. However, a localized failure as a result of large localized strains might initiate redistribution of stress-strain relationships within the body and result in a new condition of equilibriums. If this possibility actually exists, some patterns of soil loading might conceivably produce localized failures and localized yielding within the body, but not cause a general failure of the entire soil depth.

Additional considerations are introduced when the lower boundary of soil mass is in contact with a buried structure. The soil and structure, when surface load is applied, will deflect as a unit as long as contact is maintained at their interface. Each material will then contribute some portion to total soil-structure resistance. However, in much the same way that the total shearing resistance at a soil mass is not the integrated sum of peak strengths at its interior points, the maximum load resistance of the composite soil-structure system is not necessarily the sum of the individual peak resistances. A structure which will tolerate only limited deflections, when covered with a soil which requires large strains to develop peak strength, may fail under applied surface load before any appreciable soil strength can be mobilized. Conversely, a highly-flexible structure may contribute little to the composite strength of a soil-structure system containing a stiff soil. This leads to the conclusion that, if maximum resistance is to be developed for a soil-structure system, the movements at the soil-structure boundary should simultaneously develop the peak resistances of the soil and of the structure. Such optimum behavior may represent an unrealistic design objective, however.

As indicated earlier, the portion of the applied surface load supported by the buried panels of the test series is measured by the observed-load factor, expressed as

$$\frac{w(r_{\Delta}, r_b)}{w(r_{\Delta}, r_b) + w\,\underline{soil}} .$$

Applying this to an actual buried structure, several design alternatives are suggested. As one possibility, consider the case where cover soils have little strength, burial depths are very small, and the structure and its foundation will be very rigid in comparison with the cover soil. For this situation, the term w soil in the denominator may be of little consequence, and the observed-load factor can be approximated as unity (note that the corresponding equivalent-load factor, representing the distribution of structural load, need not be taken as unity). At the other extreme, when appreciable burial depths are contemplated in a soil of good strength properties, use of a very flexible structure of limited strength could be considered. It would appear, from cursory examination of the terms in the observed-load factor, that a structure of essentially no strength might be feasible at large burial depths. Although this might prove to be true, there may also exist a potential for localized soil failures at finite depths, probably associated with large structural deformations. Should such a condition actually exist, some finite level of structural resistance would be advantageous to inhibit localized soil failures at depths within the soil mass.

These concepts can now be extended to soils with idealized strength properties. First, consider a saturated clay whose shearing strength is related only to choesion and is constant at all points within the soil body. Finite shearing strains are developed in the soil as surface load is applied, developing a finite load resistance for the cover soil. However, for a given depth of soil, the ultimate load which the soil can support is its total effective shearing resistance along its incipient planes of sliding. These planes, as previously stated, are assumed to be vertical upward extensions from the periphery of a buried structure. The shearing resistance of a fixed loaded area of soil, for this idealized situation, would vary linearly with depth of burial but remain independent of load. In the two-dimensional case, by this same reasoning, the total resistance of a clay cover soil becomes a linear function of the burial depth ratio. Thus, the observed-load factor associated with fully-mobilized resistances of ideal clays above a buried structural panel may be written as:

$$\frac{w(r_\Delta, r_b)}{w(r_\Delta, r_b) + f(\text{depth})} .$$

The load supported by the buried structure, which appears as the $w(r_\Delta, r_b)$ term in this expression for observed-load factor, is related to the vertical displacement at the soil-structure interface. (This load-deflection relationship was, for the buried test panel, observed to be linear at zero burial depth but non-linear at depths. The load redistribution index I_r supplied an indication of the degree of non-linearity). Momentarily neglecting the resistance of the structure itself, this displacement may be considered as the net soil strain at the interface when there is incipient failure of the loaded soil mass above the buried structure. Utilizing the previously stated assumption, it was concluded that the total load resistance of this idealized soil mass would be a linear function of the burial depth ratio. From this, the net displacement at the soil-structure interface, considering only the soil, may also be approximated as a linear function of the burial depth ratio. The soil and structure act in combination to resist load and, for these idealized conditions, it appears that the observed load on the structure should increase in a roughly linear fashion with depth. If such is the case, there would belittle change in the observed-load factor

$$\frac{w(r_\Delta, r_b)}{w(r_\Delta, r_b) + w\,\underline{\text{soil}}}$$

as burial depth ratios are increased in an ideal clay.

Next, assume an ideal fully-drained granular soil, whose shearing strength is related to applied load and to its effective angle of internal friction. Again postulating that ultimate failure will be accompanied by vertical sliding planes between the panel periphery and the ground surface, and still assuming that panel deflections are adequate to mobilize this ultimate soil resistance, the total load-resistance of the soil mass is no longer independent of applied surface load. The expression for observed-load factor at a specified depth of cover now becomes

$$\frac{w(r_\Delta, r_b)}{w(r_\Delta, r_b) + f(\text{depth})^n} ,$$

where n is some factor larger than unity. This suggests that the observed-load factor should, for an ideal sand, decrease with increasing burial depths.

Increasing the panel deflection ratio would, for an ideal sand, cease to be effective once the soil had yielded sufficiently to develop its ultimate sliding resistance. The relative density of such soils could conceivably become important, since the panel deflection necessary to mobilize the full shearing resistance of a loose sand might exceed permissible structural limits. At the other extreme, a dense granular soil could develop such small strains under applied loads that only a portion of the potential panel resistance is mobilized prior to soil failure. Immediately prior to failure, however, a soil of this type might expand and thus induce intolerably-large panel deflections.

The individual test series are now separately discussed, and their results briefly compared.

1. Sand, Static 2-D Loading - Load factors are plotted in Figures A-5 to A-8, inclusive. Both plots decreased rapidly as the panel burial ratio increased to 1.0 or 1.5, then exhibited an appreciably flatter slope to the test limit, $r_b = 4.5$. Varying the panel deflection ratio from 0.025 to 0.100 had little effect on either load factor. There was evidence of a non-uniform distribution of panel load since (as was found to be the case in all test series) the equivalent-load factors were less than observed-load factors. This indicated a concentration of panel loading adjacent to panel supports.

2. Sand, Static 3-D Loading - Load factors are plotted in Figures A-9 to A-11, inclusive. Again, both load-factors plots decreased rapidly until the panel burial ratio reached 1.0, then displayed a flatter slope to the test limit of $r_b = 2.0$. However, increasing the panel deflection ratio from 0.025 to 0.075 reduced the observed-load factor, particularly for the larger values of r_b. This suggests that the shearing resistance of the soil was not fully mobilized by the smaller panel deflection.

For $r_b = 1.0$ and $r_\Delta = 0.025$ the observed-load factor is larger in the 3-D test than in the 2-D series, while the equivalent-load factors are roughly equal. For larger panel deflections ($r_\Delta = 0.050$ and 0.075) both factors become increasingly greater than in the 2-D series. The data also indicate that a more favorable distribution of panel load is realized in the three-dimensional

tests. This is inferred from the greater divergence of the curves of equivalent-load factor and observed-load factor, as compared to the two-dimensional tests. It was also observed that ability of the soil to resist load was greater in the 3-D tests. This suggests that end-effects adjacent to the panel were actually of significance in the 3-D tests despite the deliberate use of an elongated test panel. In any event, it appears that any frictional forces which may be developed at the face of the glass-box in the 2-D tests are soon over-shadowed by three-dimensional effects in the 3-D series.

3. Sand (dense), Static 2-D Loading - Load factors are plotted in Figures A-12 and A-13. These were computed from the results of an earlier test series, which used a dense sand[9], and have been included for comparison. The panel used in this series had a deflection ratio of 0.025 under a 0.100 psi uniform load, while the panels used in all later 2-D series required a 0.153 psi load for the same deflection ratio. Thus, any direct comparisons between the results for the medium sand (Figures A-5 and A-6) and the dense sand (Figures A-12 and A-13) become somewhat obscured. Also, only equivalent-load factors could be computed, since actual panel loads were not observed. However, it is of interest to note that the load factor curves for the dense sand are quite similar in shape to those for the medium sand. Absolute values of the equivalent-load factor are somewhat less for the dense sand.

A distinctive feature of the 2-D tests in dense sand was the sudden collapse of the test panel for $r_b = 1.0$ and with r_Δ between 0.05 and 0.075. This effect, which did not occur in the medium sand, suggests that caution must be used when predicting soil-structure relationships for a flexible structure in a stiff soil. The abrupt failures in the dense sand were apparently accompanied by local dilation of the soil, since the areas of reference grids in the vicinity of the panel were observed to increase. (Figure A-14).

4. Sand, Dynamic 2-D Loading - Load factors are plotted in Figure A-15. The observed-load factor decreased rapidly to about 0.13 at a panel burial ratio of 0.5, and subsequently continued to decrease to a value of about 0.04 at $r_b = 2.0$, the limit of the test series. This latter value of the observed-load factor was appreciably less than the 0.10 value observed in the static tests, under otherwise comparable conditions. There was also some reduction

in the equivalent-load factor for the 3-D tests, as compared with the 2-D series, but this effect was less striking.

As earlier explained, the applied surface load was held constant for the dynamic series. The maximum panel deflection was measured, and the load-factors computed by equating $W(r_\Delta, 0)$ to the product of panel area and peak dynamic pressure. Thus, the load factors were actually computed for different panel deflection ratios, although they are presented in Figure A-15 as a continuous plot. Also, the use of the peak dynamic pressure in calculating $W(r_\Delta, 0)$ may have resulted in load factors which are, in fact, too low.

5. Clay, Static 2-D Loading - Load factors are plotted in Figures A-16 to A-19, inclusive. The plot of observed-load factors decreased rapidly as panel burial ratios increased to 1.0 or 1.5, after which the rate of decrease lessened. However, with the panel burial ratio held constant, each successive increase in panel deflection ratio was accompanied by a related increase in the observed-load factor. This suggests that the shearing strength of a fixed depth of soil increased little, if at all, as the applied surface load was increased. It may be recalled, as illustrated by Figures A-5 to A-8, that the observed-load factor in similar tests on a granular soil remained essentially constant, suggesting a linear relationship between soil shearing strength and applied surface load for a fixed burial depth ratio.

The equivalent load factor showed a marked reduction as the panel burial ratio increased to 0.5. For increasing values of r_b, up to the series limit of $r_b = 2.0$, the equivalent-load factor remained essentially constant. Thus, at the larger burial ratios, the equivalent-load plot began to parallel the observed-load plot. There appeared to be some minor increase in the equivalent-load factor as r_Δ increased, although this trend was uncertain.

It was observed that, as loading progressed, the central portion of the panel deflected until contact with the cover soil was lost. Obviously, particularly at low burial ratios, a larger portion of the panel load was concentrated near the supported edges. Thus, for $r_\Delta = 0.10$ and $r_b = 0.5$, the observed-load factor and the equivalent-load factor were 0.68 and 0.08, respectively. At $r_b = 2.0$, holding r_Δ unchanged, corresponding load factor values were 0.23 and 0.035.

Observed-load factors, regardless of the values of r_b and r_Δ at which comparisons are made, were roughly twice as large for 2-D tests in clay as for 2-D tests in sand. This disparity, understandably, was greatest for the larger values of r_Δ . However, there was very little difference between values of the equivalent-load factors for comparable test conditions in the two soils.

After the surface load was applied to the test-specimen, readings were taken of panel load and panel deflection. Since panel deflections continued to increase under a given load increment, although at a decreasing rate, a portion of the observed panel deflection was the result of time-dependent soil deformation. Recognizing this effect, a standardized test procedure was adopted whereby deflection measurements were made within a controlled time after load application; this procedure did not, however, exclude a cumulative effect on measured panel deflections as load increments were increased.

6. Clay, Dynamic 2-D Loading - Figure A-20 shows plotted load factors for 2-D dynamic tests using clay soil. Both load factors decreased rapidly with increasing panel burial depth ratios, up to the test limits of $r_b = 1.0$. The two plots remained essentially parallel throughout their lengths, indicating a constant relationship between panel load distribution and applied load. The observed-load factor decreased from 0.32 at $r_b = 0.5$ to 0.06 at $r_b = 1.0$. In this same interval, the effective-load factor decreased from 0.10 to 0.01.

The load factors for the clay dynamic-load series were appreciably less than those computed for the clay static-load series. In fact, particularly as r_b approached 1.0, the dynamic load-factors for the clay were very similar to the dynamic load-factors for sand (Figure A-13).

7. General Discussion of Load Redistribution - Figures A-21 through A-28 present, for all series of tests, the structural load redistribution ratios as related to burial ratios. These results, since they are considered particularly significant, are discussed separately from the remainder of the test data.

Figures A-21 through A-24 show data obtained from the two-dimensional tests in both sand and clay. The load redistribution for both

soil types is seen to increase quite rapidly with burial ratio, up to a depth of approximately one panel width. The redistribution ratio for the cohesionless sand soil then remains essential constant as burial depth increases, especially for deflection ratios greater than 0.025. For the cohesive soil, however, this ratio reaches a maximum value and subsequently decreases. The plots of observed and equivalent load factors were also seen to experience major changes in slope at burial ratios of 1.0 to 1.5. Thus, as the burial ratio increases from zero to 1.0 or 1.5, greater ability of the soil-structure system to withstand surface loading is evidenced. This gain results both from a favorable redistribution of the load transmitted to the panel and from the continuing mobilization of soil shearing strength. At greater depths, however, it appears that the trend towards favorable load redistribution no longer exists. As a consequence, the rate of change of load factors reduces.

In cohesionless soil, as was previously postulated, a constant redistribution of load could accompany an increasing burial depth if localized yielding in the soil body established the limiting soil resistance. Once sufficient soil cover is obtained to allow formation of this localized failure, the effective soil resistance would become essentially independent of additional soil cover. While there is an appreciable scatter in the data, the curves for the larger deflection ratios tend to indicate that the load redistribution ratio will also become constant in the cohesive soil with increasing depth of burial. This might be expected if, at some limiting depth, the soil cover functions as a very deep beam whose strength is limited by its shearing resistance.

Figures A-25, A-26 and A-27 present similar results for the three-dimensional tests in sand. While the same trends are seen as in the two-dimensional tests, greater load redistribution effects are evidenced.

Figure A-28 presents results of the dynamic tests in both the sand and clay soils. Here the effects of redistribution at shallow depths of burial are less pronounced than for static surface loading. This might be due to the initial impulse loading as the shock pulse engulfs the structure. However, once the panel begins to deform, accompanying deformations of the soil must occur if the loading on the structure is to be maintained. This, in turn, will mobilize the resistance of the soil to deformation.

CONCLUSIONS

The following conclusions have been postulated from the results of the exploratory tests as reported in this paper. Their validity is, in our present state of knowledge, restricted solely to the conditions of the test series. It is cautioned that any attempts to extrapolate these conclusions to other environments, in particular the evaluation of full-scale structures and actual loading conditions, should be preceded by a careful program of testing and analysis.

1. The total load actually carried by a buried flexible panel, as a consequence of applying either static or dynamic loading at the ground surface, was found to be appreciably less than the product of the applied surface pressure and the panel area. This observed effect was of particular significance at burial depths of one to two panel widths. The reduction in load occurred both in clay and in sand soils but, for the particular soils used in these tests, was found to be considerably greater for the sand.

2. Application of a uniform surface loading resulted in a non-uniform distribution of loading on a buried flexible panel. There was evidence that the panel load was concentrated near the supported panel edges, for all soils and for all conditions of loading. (Such a load distribution is, of course, favorable in the design of a flexural member). The benefits of panel load redistribution appeared most significant in clay soils and at low burial depths.

3. The contribution which the soil above a buried flexible panel made to combined soil-structure resistance appeared to be a function of applied surface load for the granular soil. In the case of the clay soil, there was evidence that a limiting soil strength existed at each burial depth and was essentially independent of applied surface load.

4. The load factors obtained in the 2-D tests were, with very minor exceptions, larger than those obtained in the 3-D tests. This suggests that the 2-D test may be used to obtain a conservative estimate of the soil strength developed in a 3-D test. This is of practical importance since the 2-D test is, in comparison with the 3-D test, inexpensive and rapid.

5. The load factors obtained in the static tests were larger than those obtained in the dynamic tests. Therefore, the results of the static tests could be used as a conservative approximation of the load reductions measured in the dynamic tests. However, the apparent significance of inertial and time-dependent effects, as observed in these tests, suggests the desirability of including dynamic loadings in any future studies.

6. An increase in the deflection of the buried panel was accompanied by an increase in the load which it supported. This increase was, for sand soils, almost proportional to the increase in applied surface load. For clay soils, however, the panel load increased more rapidly than the surface load when large panel deflections were studied. To a considerable extent, this apparent disadvantage for the clay soil was offset by a continuing trend towards more favorable distributions of panel load.

7. The relation between those strains which occur in the cover soil and the deflections in the buried flexible panel appeared, for the test conditions, to be of decided importance. Thus, as panel deflections increased in the clay loading series, the central portion of the panel lost contact with the cover soil. Also, in earlier loading tests using a dense sand, the soil was observed to undergo some local expansion immediately prior to the abrupt failure of the panel.

BIBLIOGRAPHY

1. "Design of Underground Structures to Resist Nuclear Blast", by J. L. Merritt and N. M. Newmark, Vol. 2, Final Report, Contract No. DA-49-129 eng. 312, University of Illinois for Office of Chief of Engineers, U. S. Army, Washington, D. C., April, 1958.

2. "Protective Construction Review Guide - Hardening", by Newmark, Hansen and Associates, Vol. 1, Contract No. SD-52, prepared for the Office of the Assistant Secretary of Defense, Washington, D.C., June 1961.

3. "Design of Structures to Resist Nuclear Weapons Effects", ASCE Manual of Engineering Practice No. 42, prepared by the Committee on Structural Dynamics of the Engineering Mechanics Division, 345 E. 47th St., New York, 1961.

4. "Principles and Practices for Design of Hardened Structures", Technical Documentary Report Number AFSWC-TDR-62-138, by N. M. Newmark and J. D. Haltiwanger, Contract AF-29(601)-2390, University of Illinois for Research Directorate of Air Force Special Weapons Center, Kirtland Air Force Base, New Mexico, December, 1962.

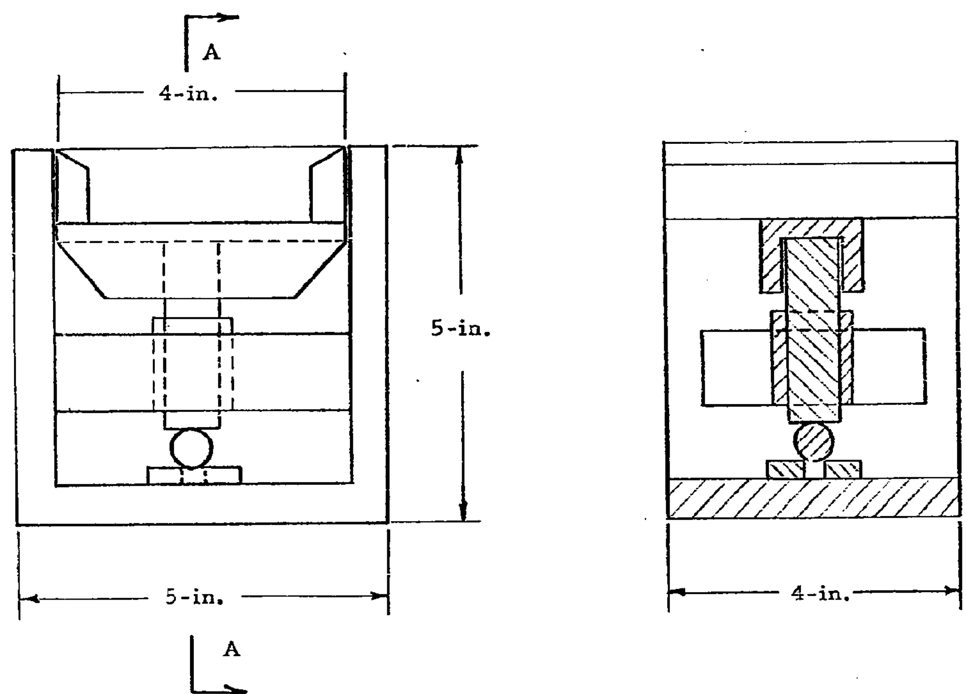
5. "Underground Structures Subject to Air Overpressure", by E. T. Selig, K. E. McKee and E. Vey, Transactions ASCE, Vol. 126, Part I, 1961.

6. "A Technique for Observing Structure-Soil Interaction", Materials Research and Standards, Vol. 1, No. 9, ASTM, September 1961.

7. "Comparative Protective Structural Design Hardening for Weapons Systems", CONFIDENTIAL, Contract No. AF29(601)-547, by Armour Research Foundation for Research Directorate of Air Force Special Weapons Center, Kirtland Air Force Base, New Mexico, June, 1960.

8. "Theoretical Soil Mechanics", by Karl Terzaghi, John Wiley and Sons, Inc., New York, 1943.

9. "Analysis of Underground Roof and Wall Panels Subjected to Nuclear Blast", CONFIDENTIAL, by E.T.Selig, Task 5, Final Report, Contract No. AF33 (600)-37360, Armour Research Foundation for Research Directorate of Air Force Special Weapons Center, Kirtland Air Force Base, New Mexico, April 1960.



Section A - A

Fig. A-1 SCHEMATIC TWO-DIMENSIONAL TEST MODEL

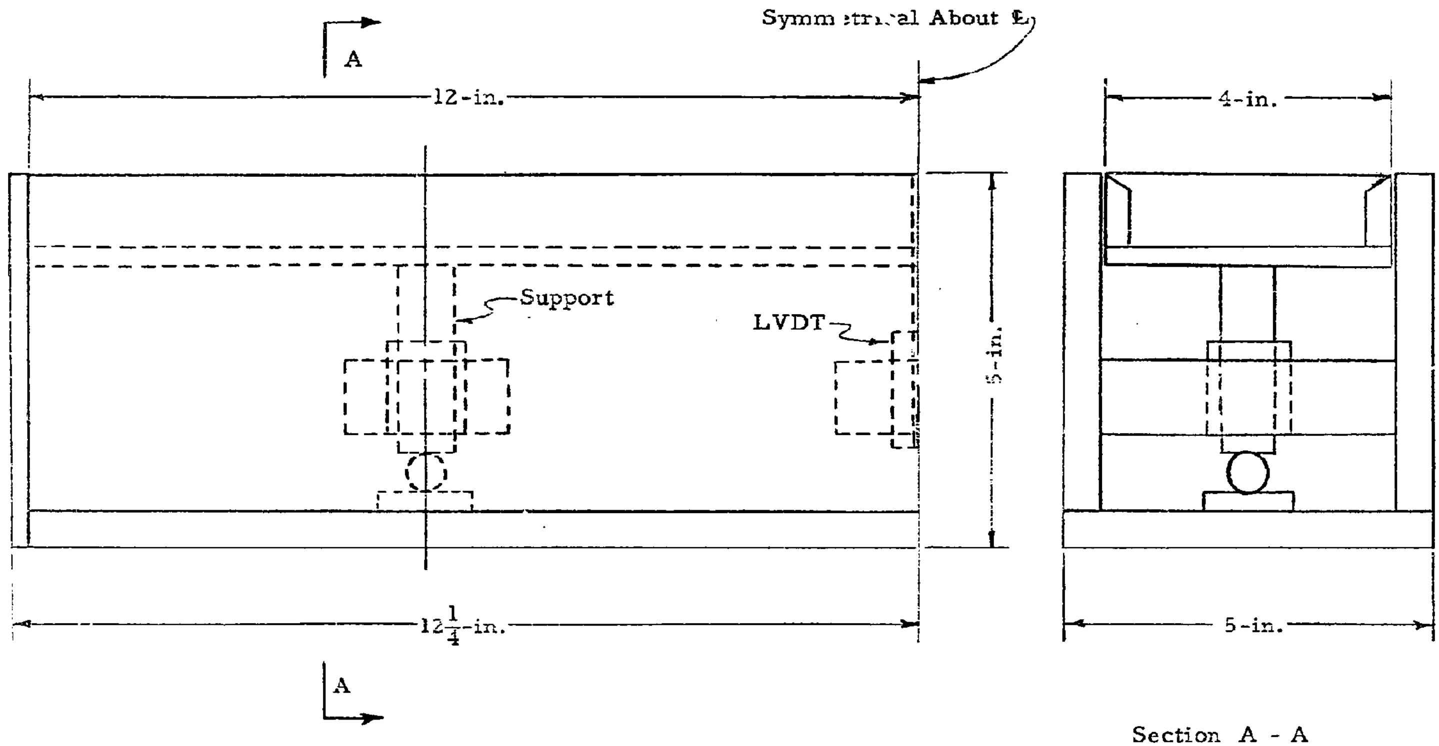


Fig. A-2 SCHEMATIC THREE-DIMENSIONAL TEST MODEL

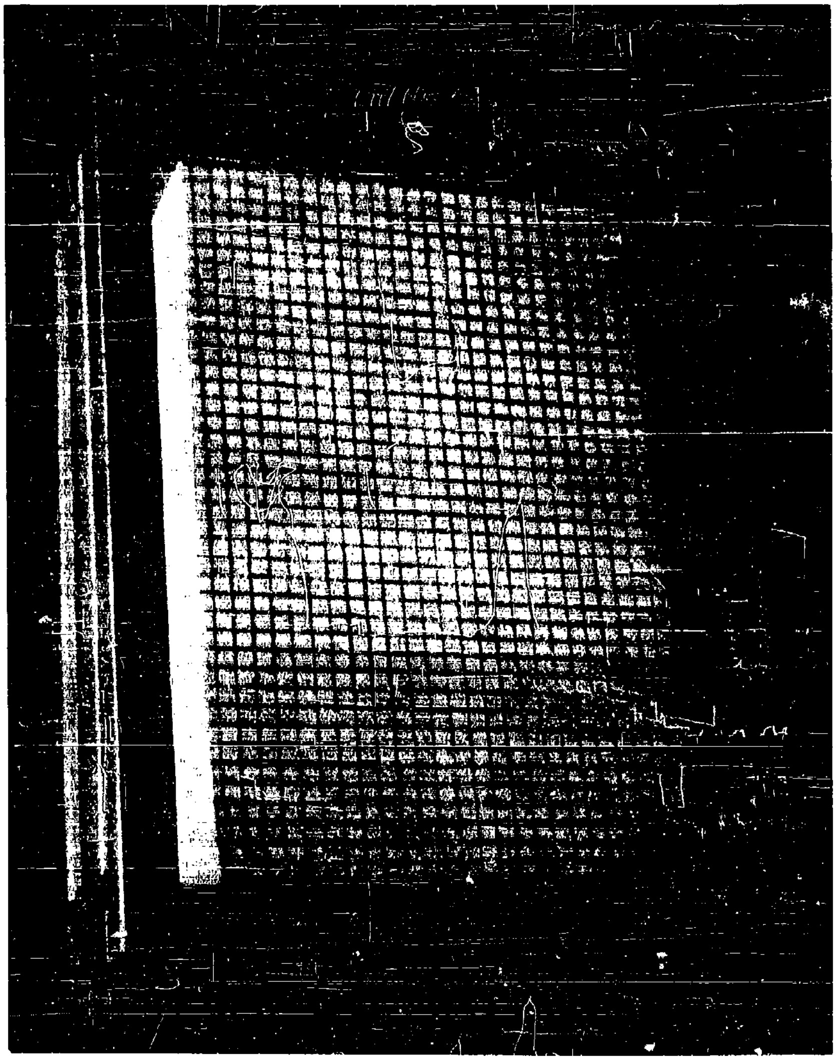


Fig. A-3 **GLASS BOX APPARATUS**

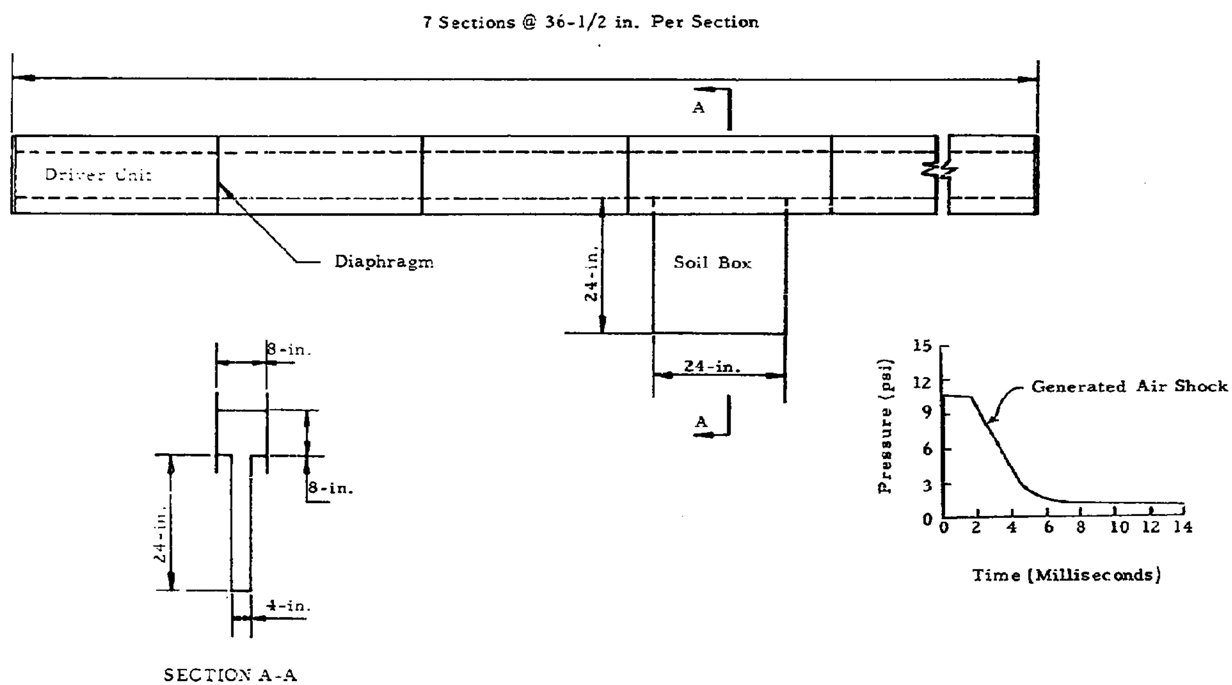


Fig. A-4 SCHEMATIC SHOCK TUBE WITH ATTACHED SOIL BOX AND GENERATED AIR SHOCK PULSE

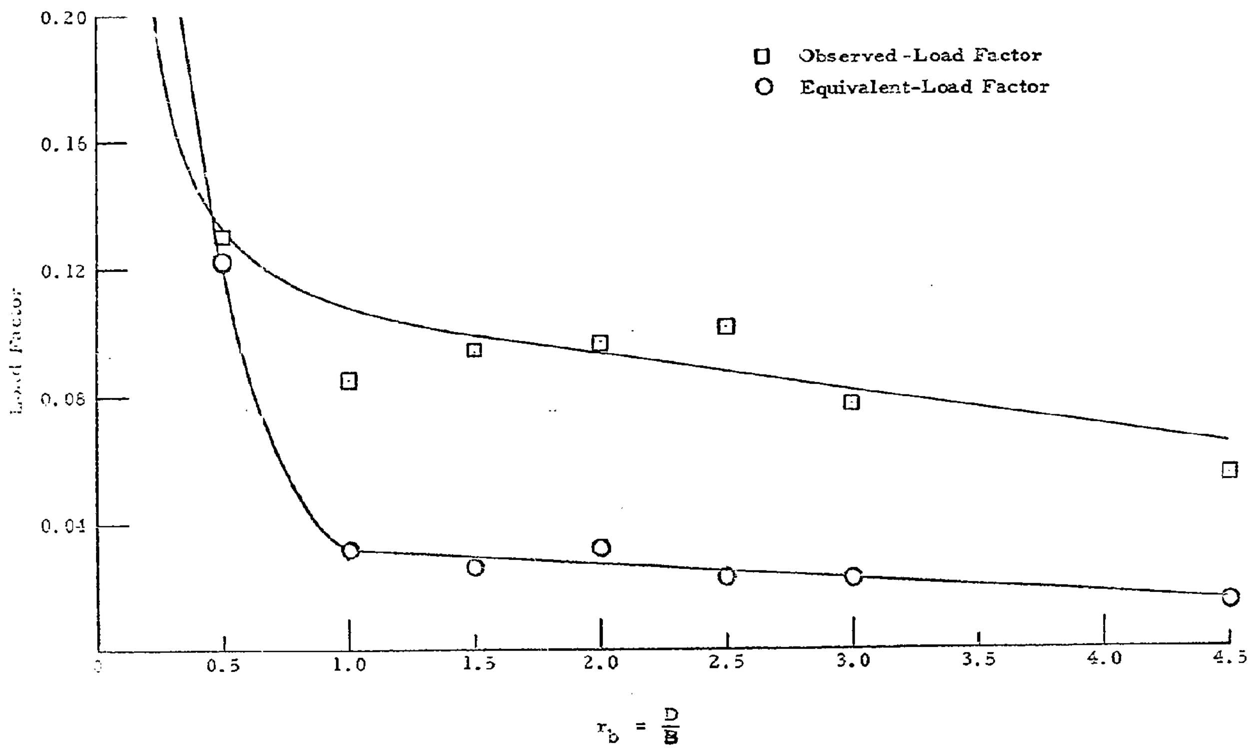


Fig. A-5 **LOAD FACTORS AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, MEDIUM DENSE SAND, r_Δ = 0.025**

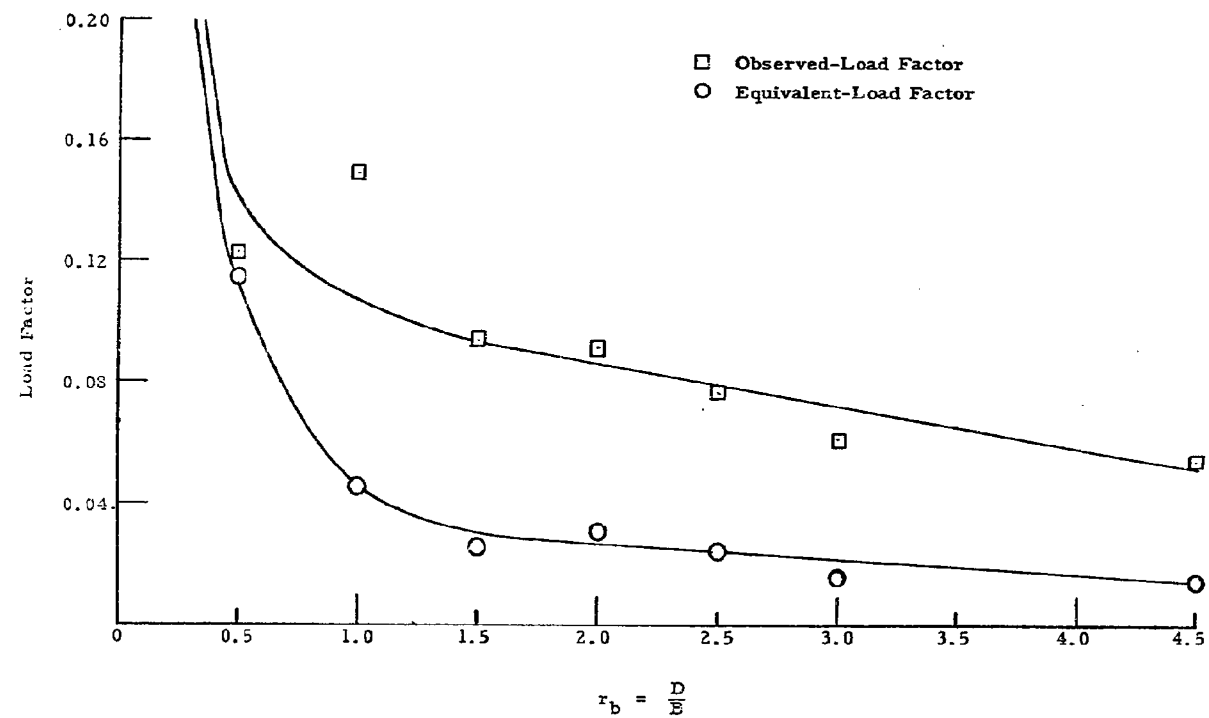


Fig. A-6 LOAD FACTORS AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, MEDIUM DENSE SAND, $r_\Delta = 0.050$

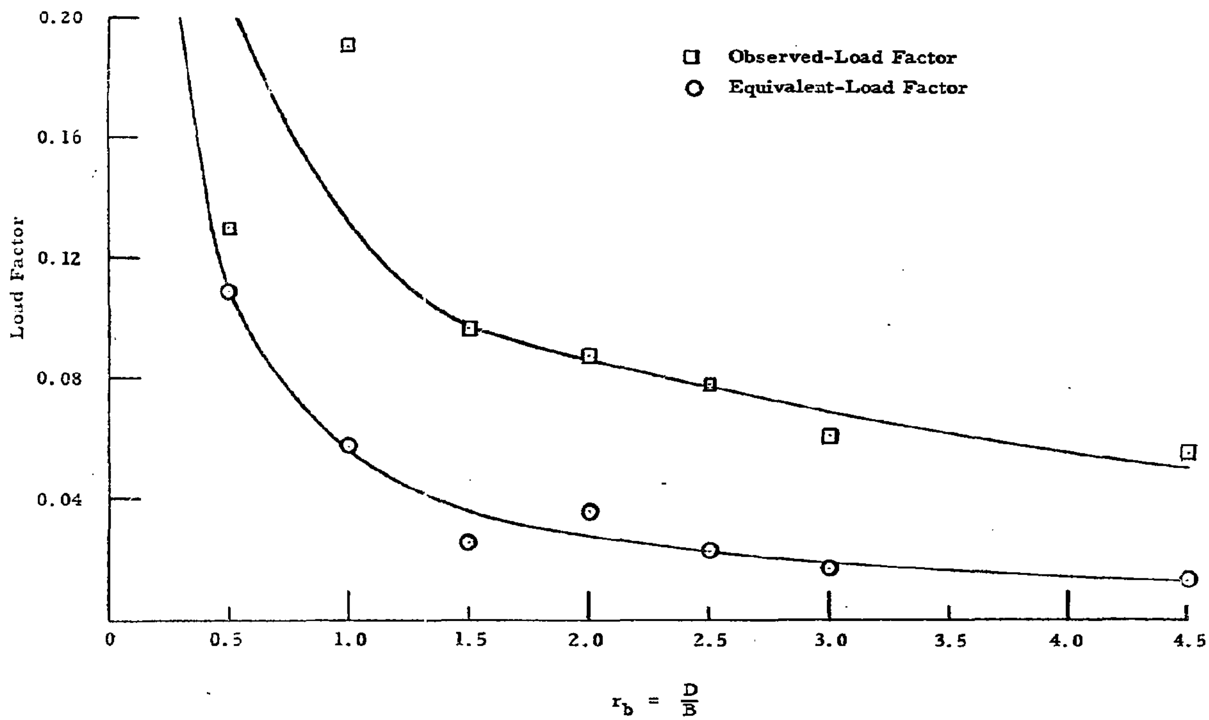


Fig. A-7 LOAD FACTORS AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, MEDIUM DENSE SAND, $r_\Delta = 0.075$

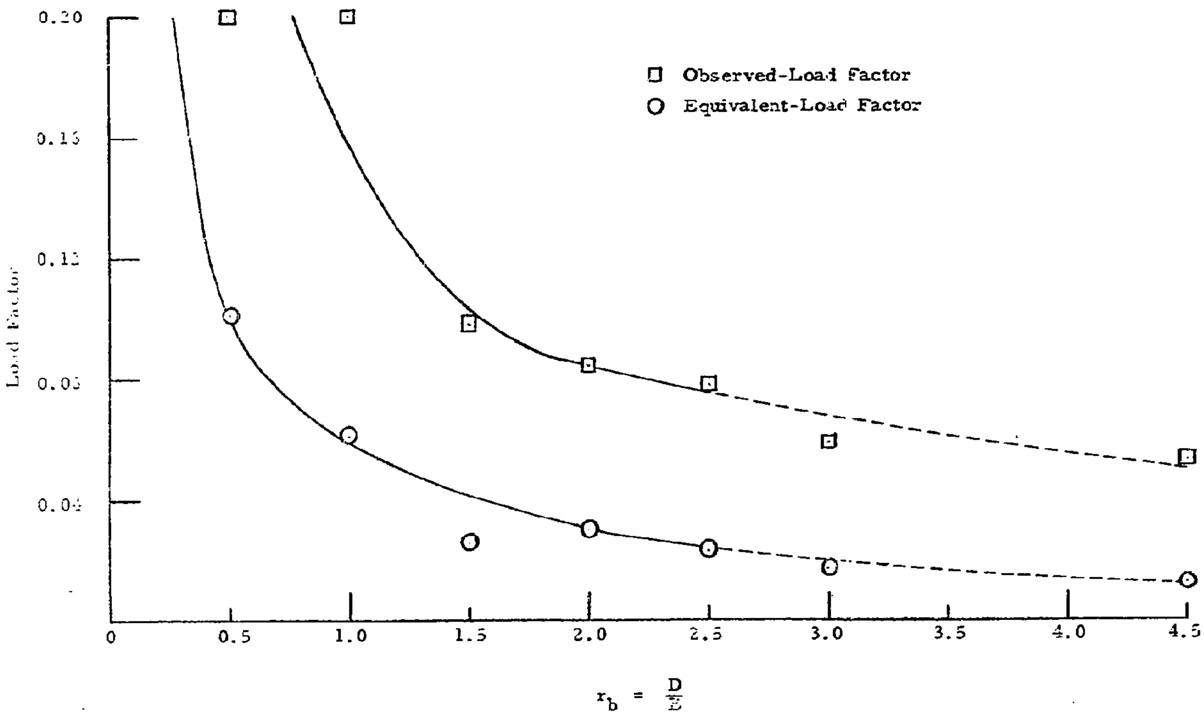


Fig. A-8 LOAD FACTORS AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, MEDIUM DENSE SAND, $r_\Delta = 0.100$

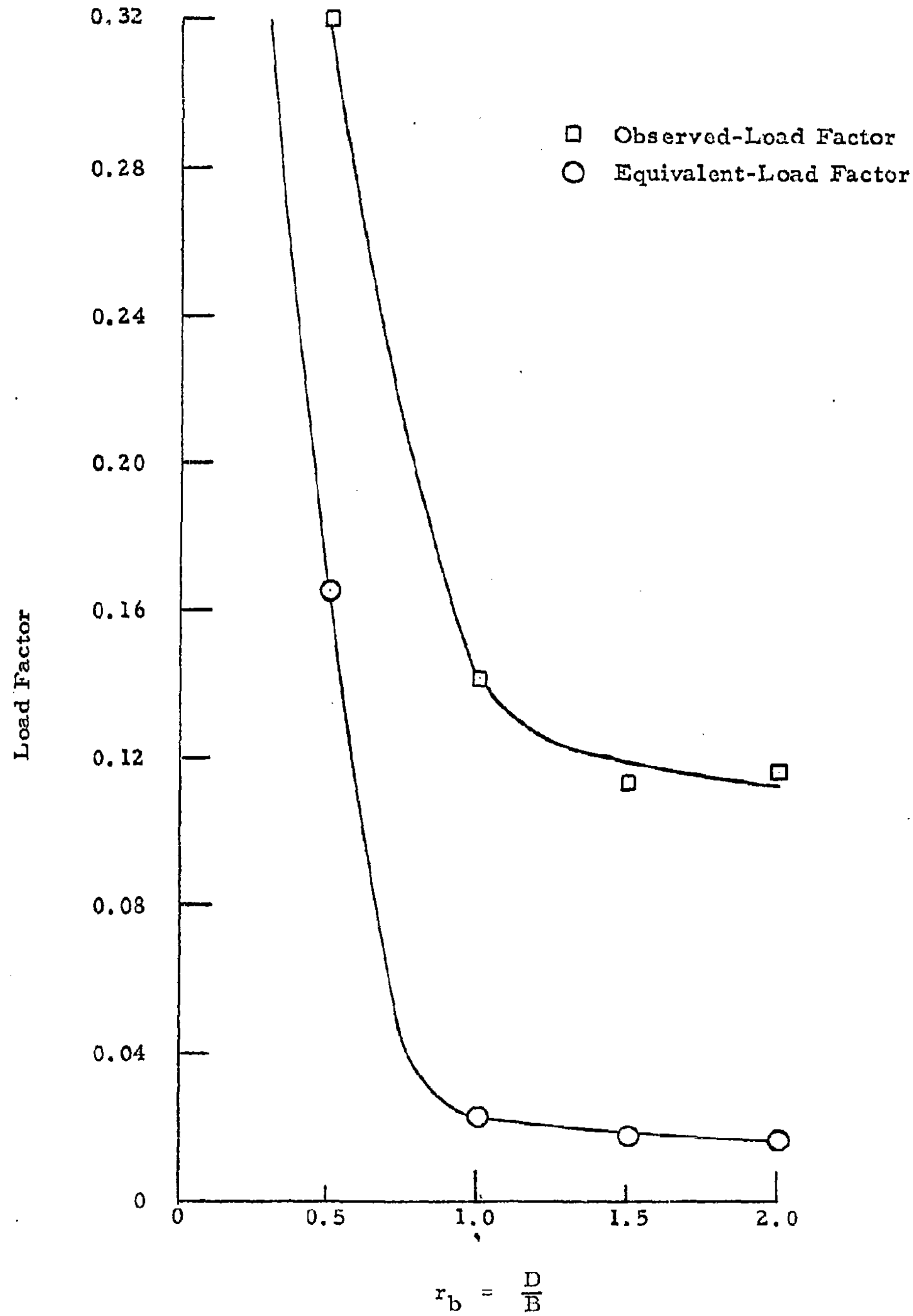


Fig. A-9 LOAD FACTORS AS RELATED TO BURIAL RATIO, THREE-DIMENSIONAL STATIC TESTS, MEDIUM DENSE SAND, $r_\Delta = 0.025$

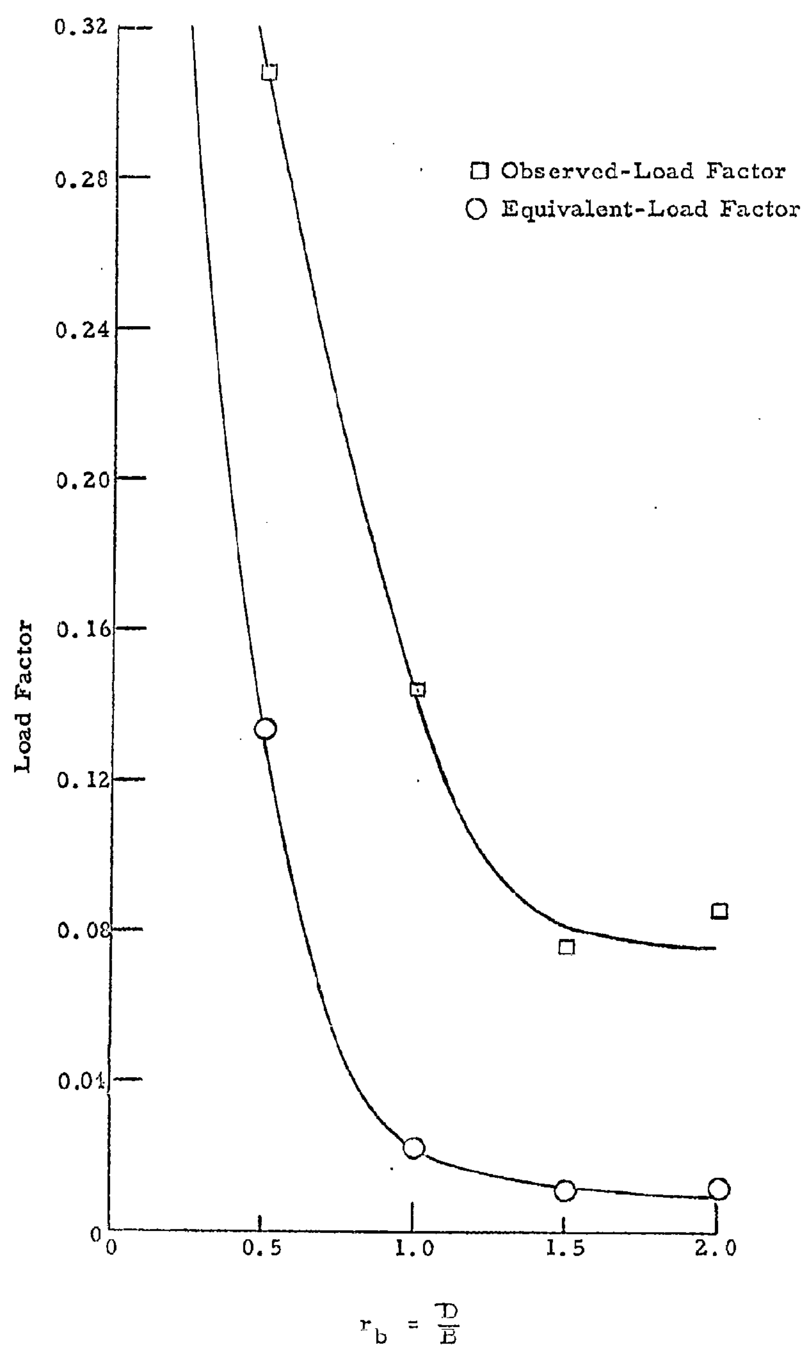


Fig. A-10 LOAD FACTORS AS RELATED TO BURIAL RATIO, THREE-DIMENSIONAL STATIC TESTS, MEDIUM DENSE SAND, r_Δ = 0.050

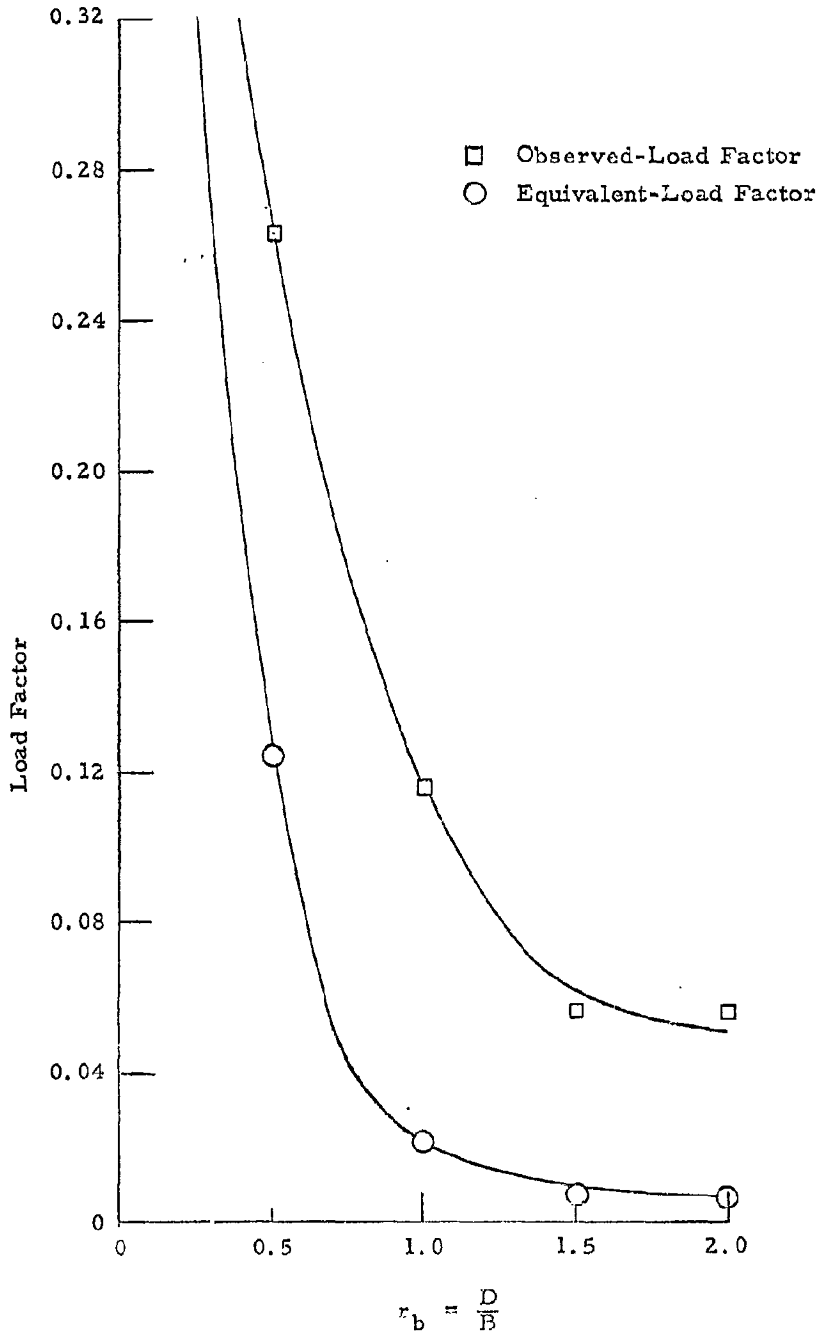


Fig. A-11 LOAD FACTORS AS RELATED TO BURIAL RATIO, THREE-DIMENSIONAL STATIC TESTS, MEDIUM DENSE SAND, $r_{\Delta} = 0.075$

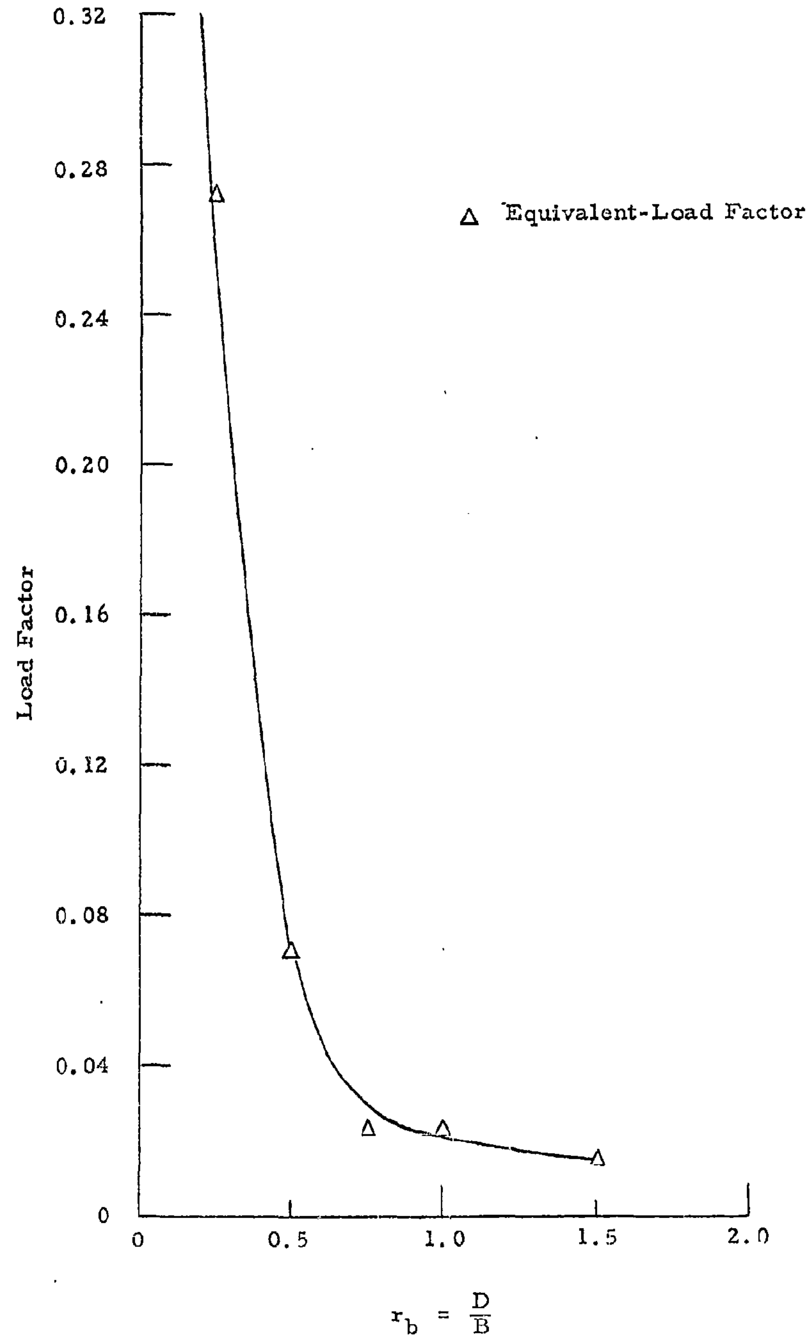


Fig. A-12 LOAD FACTORS AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, DENSE SAND, $r_{\Delta} = 0.025$

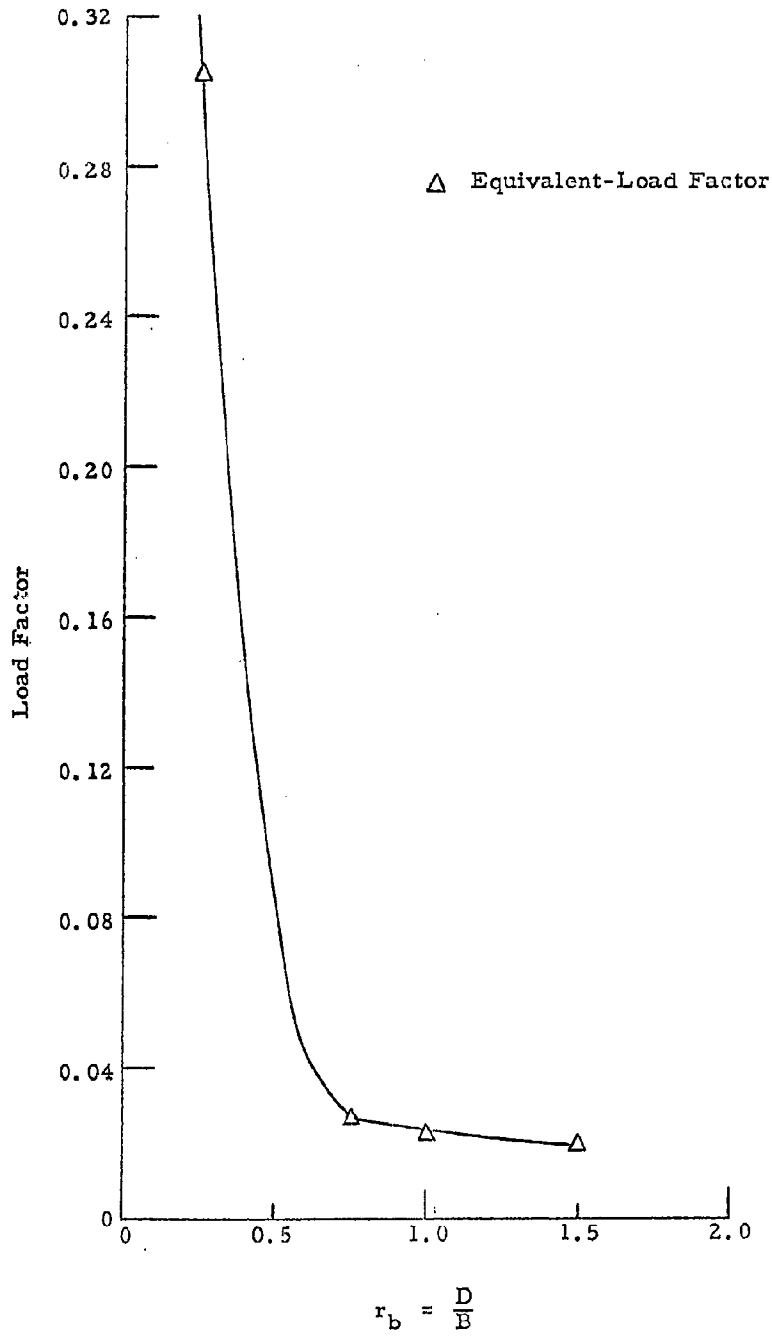


Fig. A-13 LOAD FACTORS AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, DENSE SAND, $r_{\Delta} = 0.050$

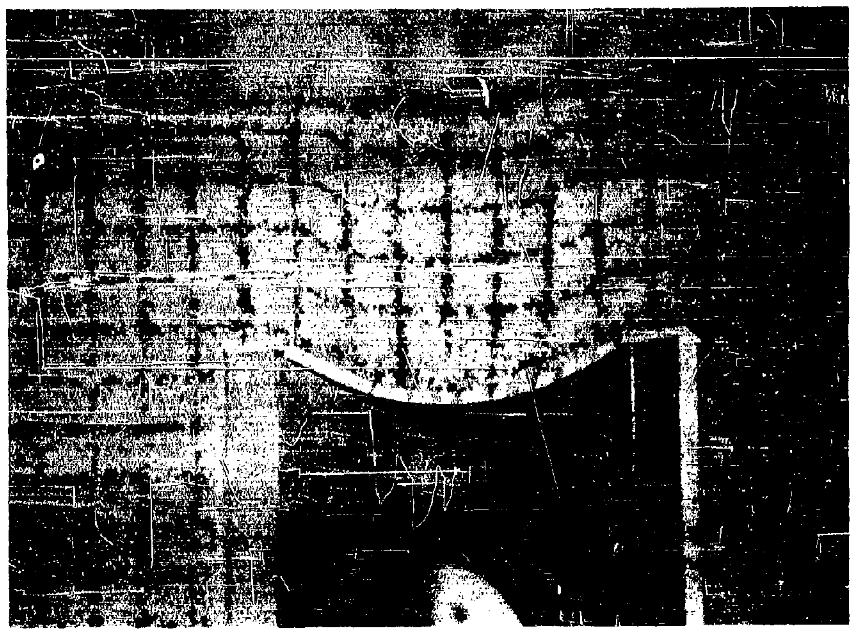


Fig. A-14 FLEXIBLE ROOF PANEL AFTER PRESSURE YIELDING

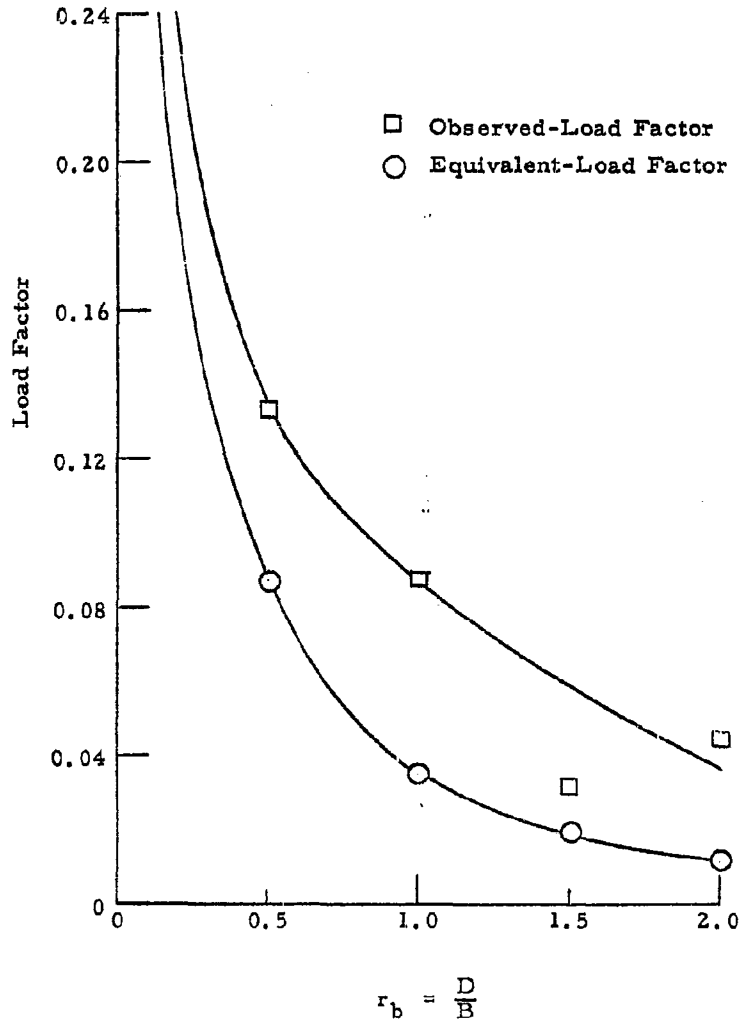


Fig. A-15 LOAD FACTORS AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL DYNAMIC TESTS, MEDIUM DENSE SAND

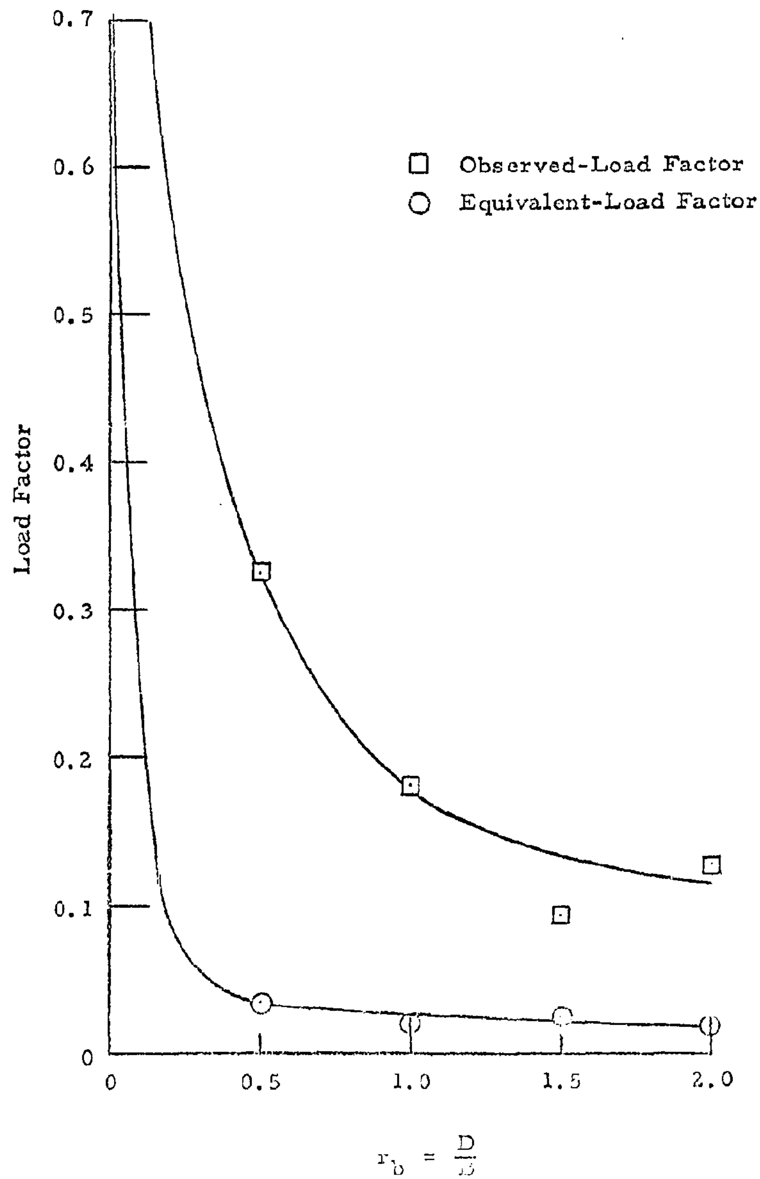


Fig. A-16 LOAD FACTORS AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, CLAY, r_Δ = 0.025

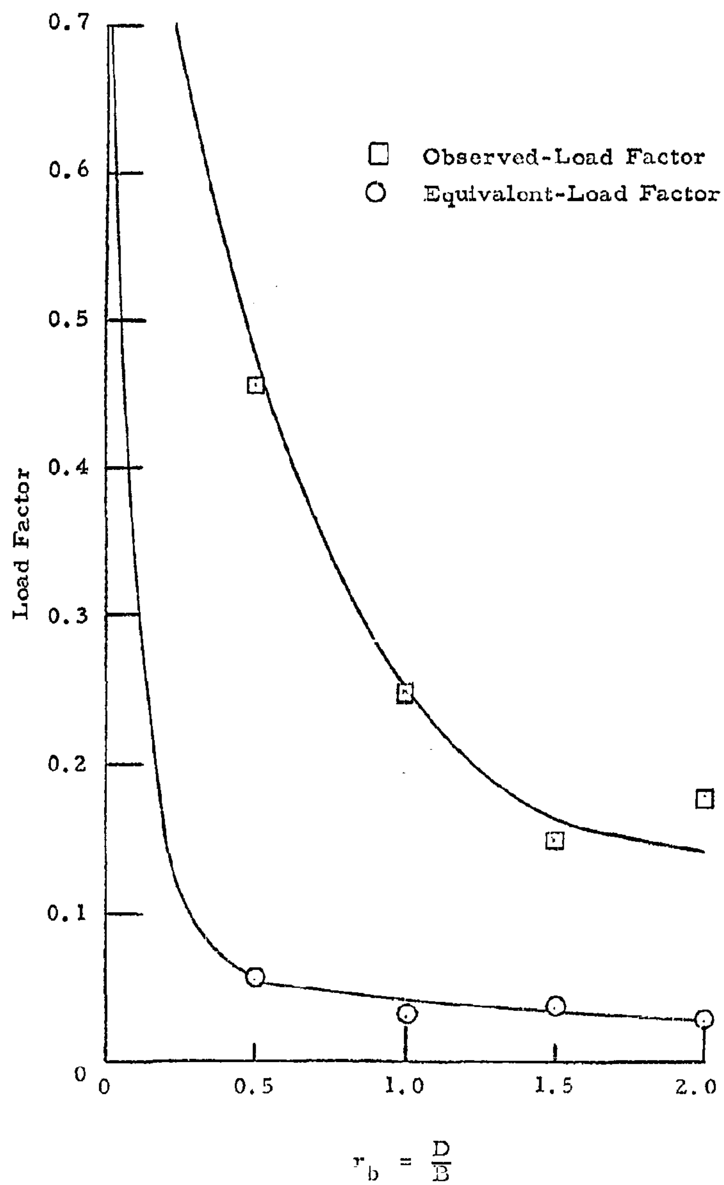


Fig. A-17 LOAD FACTORS AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, CLAY, $r_\Delta = 0.050$

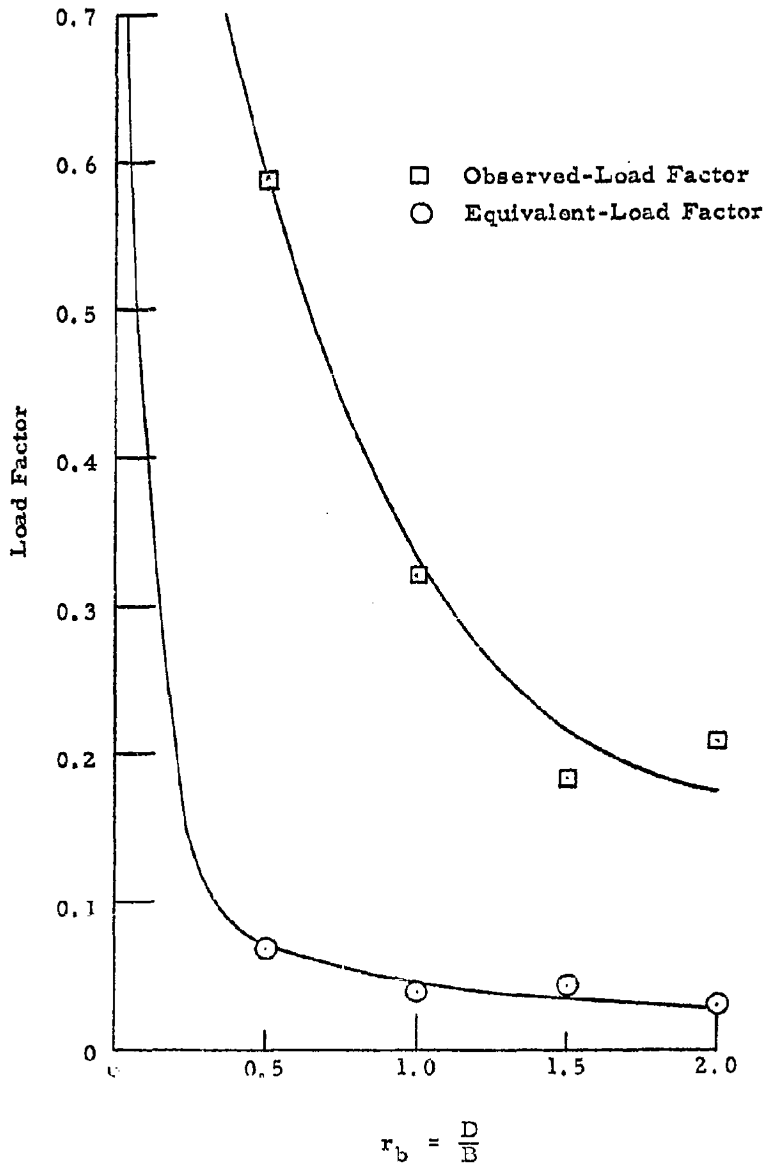
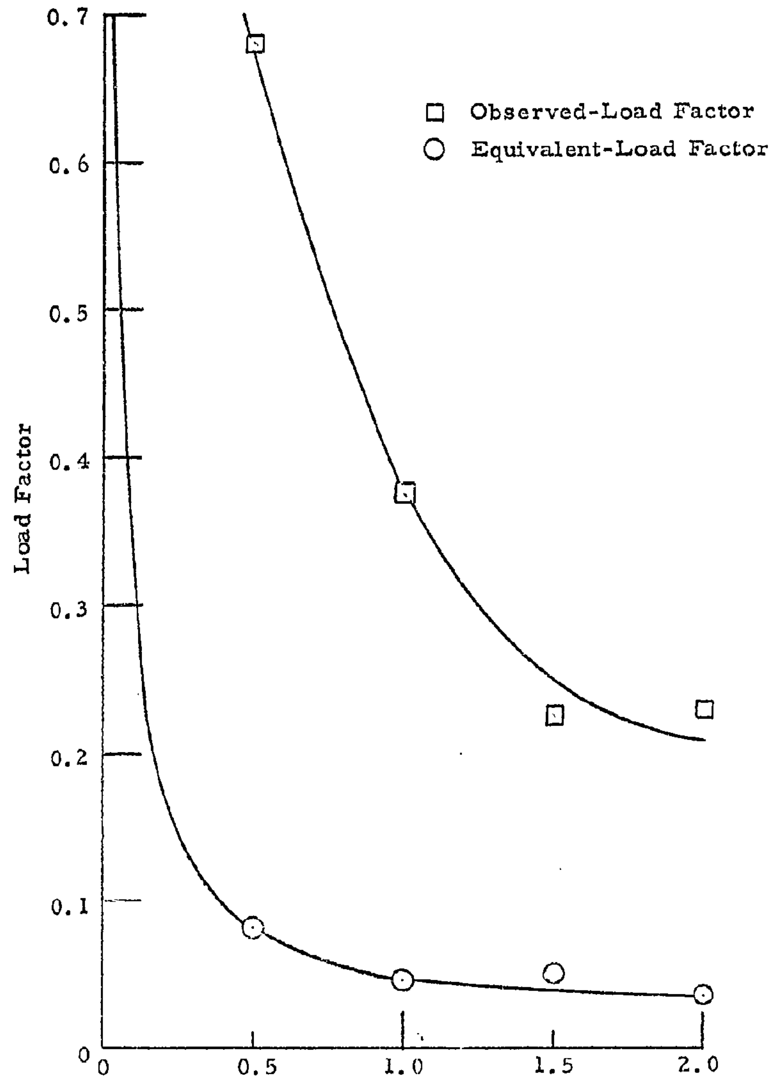


Fig. A-18 LOAD FACTORS AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, CLAY, $r_\Delta = 0.075$



$r_b = \frac{D}{B}$

Fig. A-19 LOAD FACTORS AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, CLAY, $r_\Delta = 0.100$

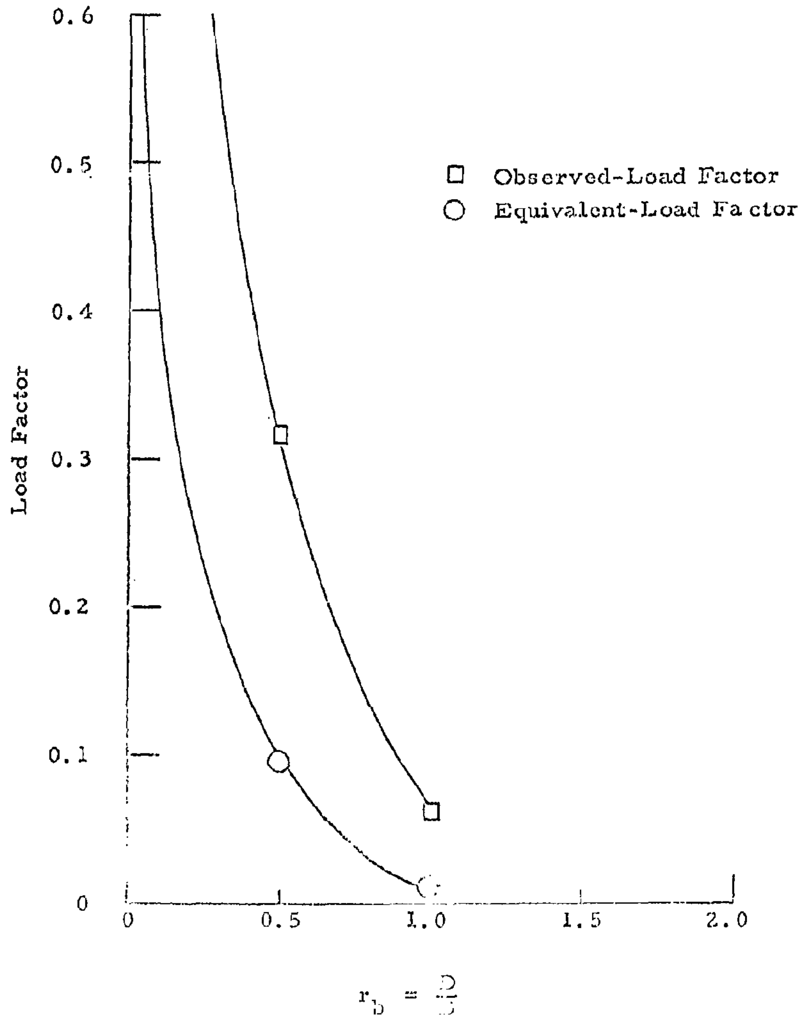


Fig. A-20 LOAD FACTORS AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL DYNAMIC TESTS, CLAY

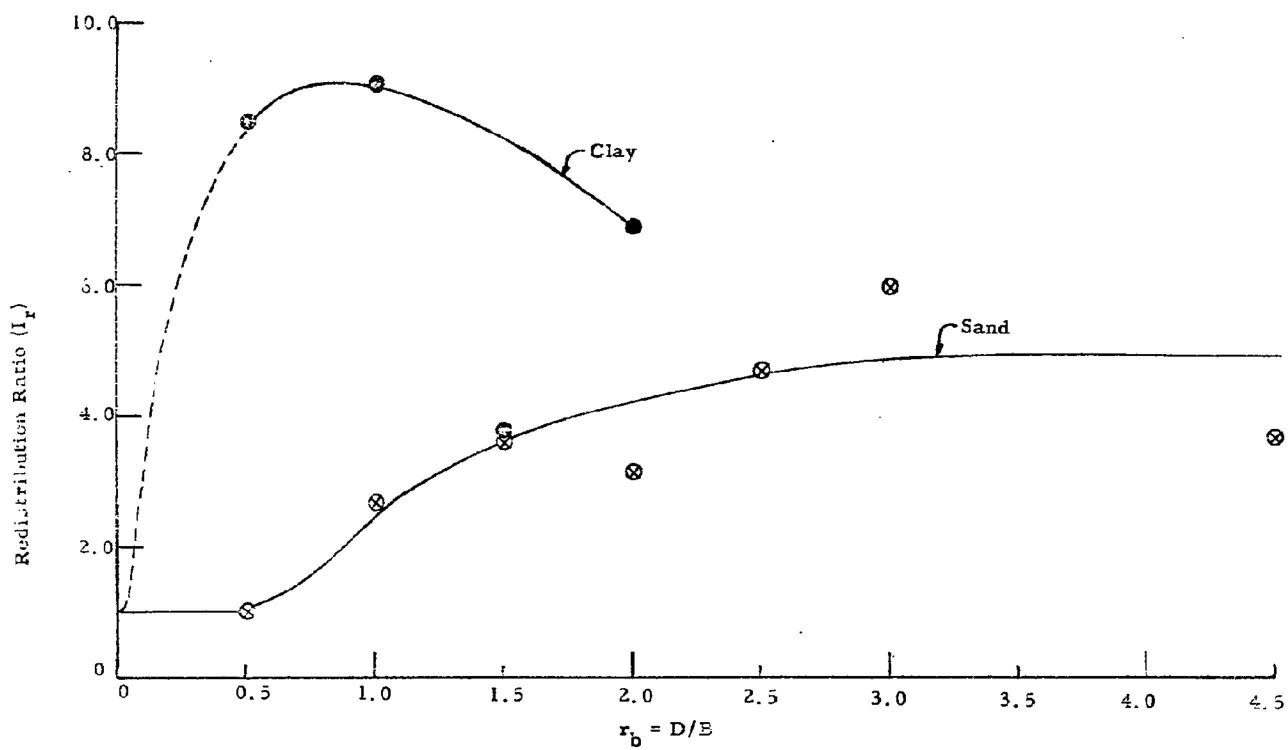


Fig. A-21 STRUCTURAL LOAD REDISTRIBUTION RATIO AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, $r_\Delta = 0.025$

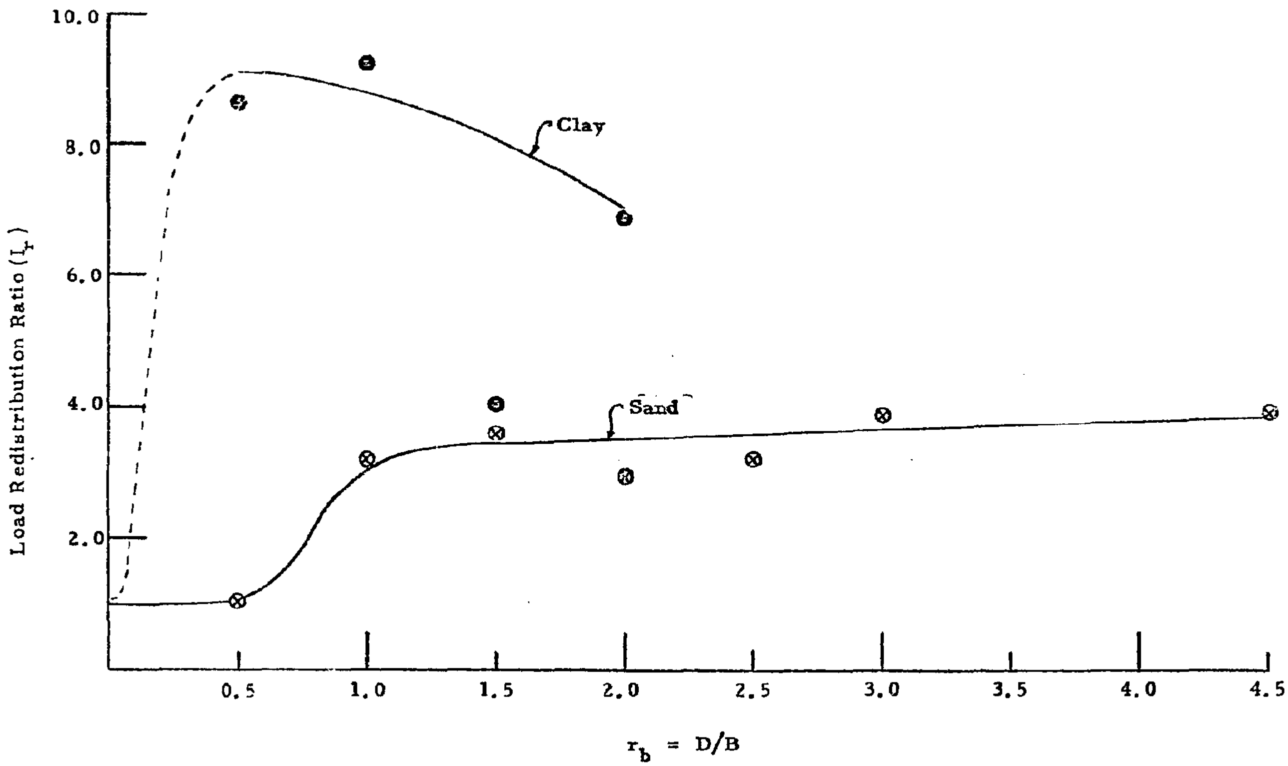


Fig. A-22 STRUCTURAL LOAD REDISTRIBUTION RATIO AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, $r_\Delta = 0.050$

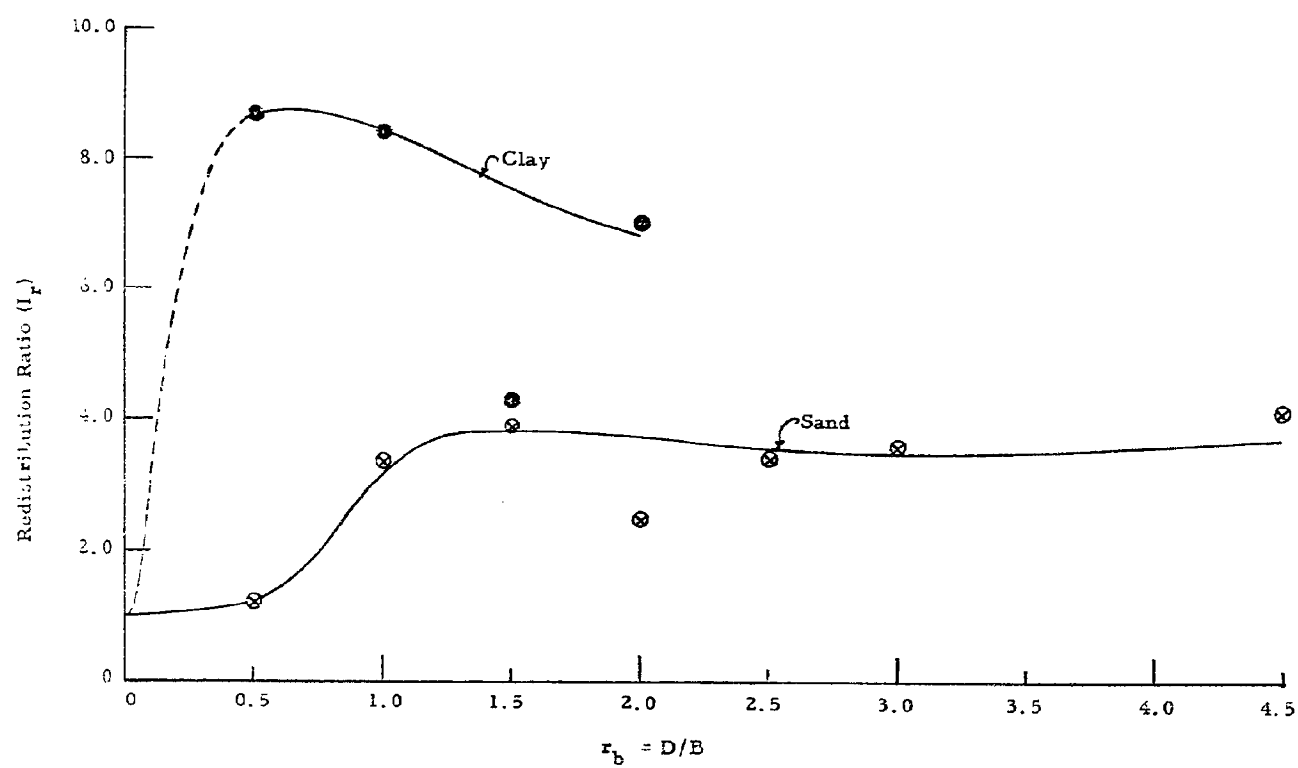


Fig. A-23 STRUCTURAL LOAD REDISTRIBUTION RATIO AS RELATED TO BURIAL RATIO TWO-DIMENSIONAL STATIC TESTS, $r_\Delta = 0.075$

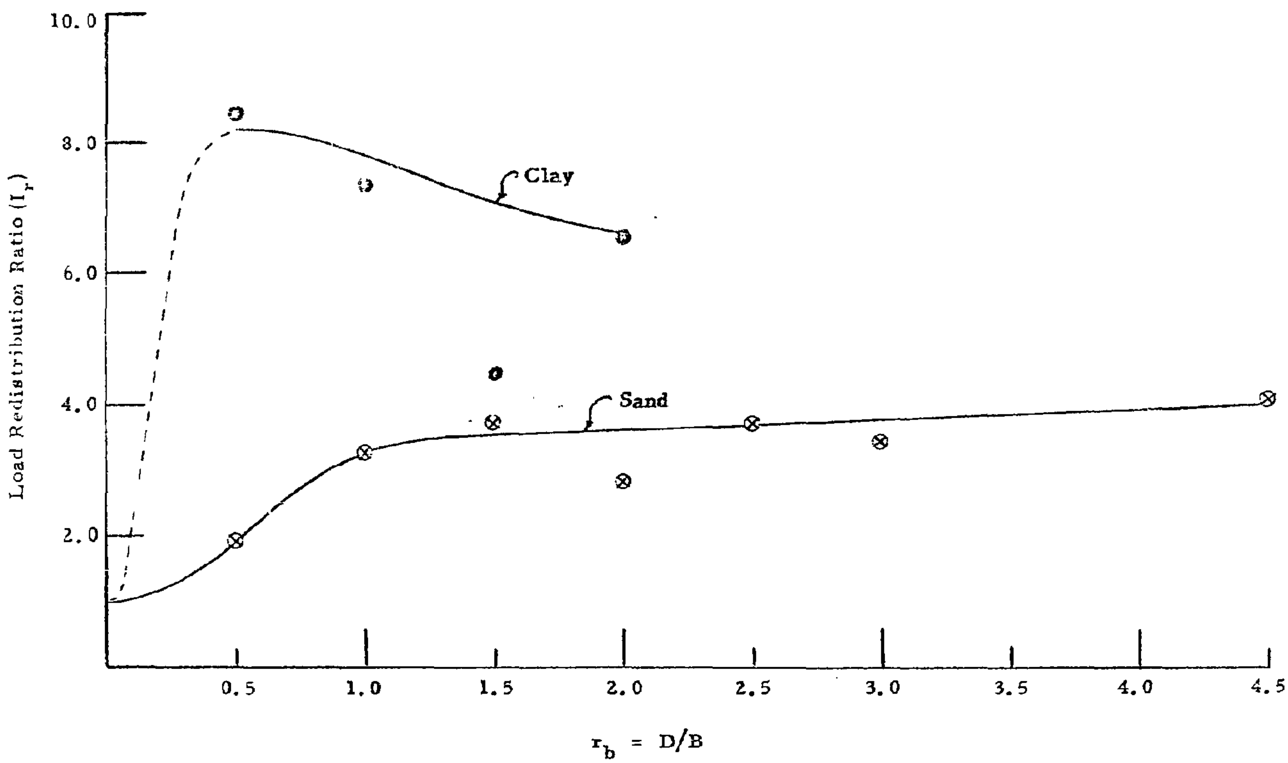


Fig. A-24 **STRUCTURAL LOAD REDISTRIBUTION RATIC AS RELATED TO BURIAL RATIO, TWO-DIMENSIONAL STATIC TESTS, r_Δ = 0.100**

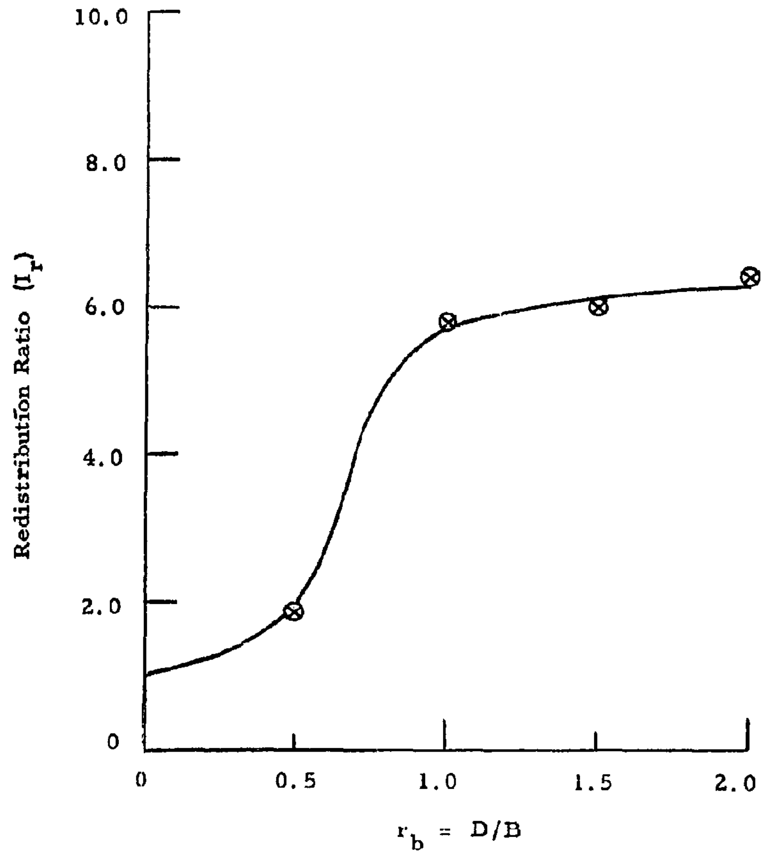


Fig. A-25 STRUCTURAL LOAD REDISTRIBUTION RATIO AS RELATED TO BURIAL RATIO, THREE DIMENSIONAL STATIC TESTS, MEDIUM DENSE SAND, $r_{\Delta} = 0.025$

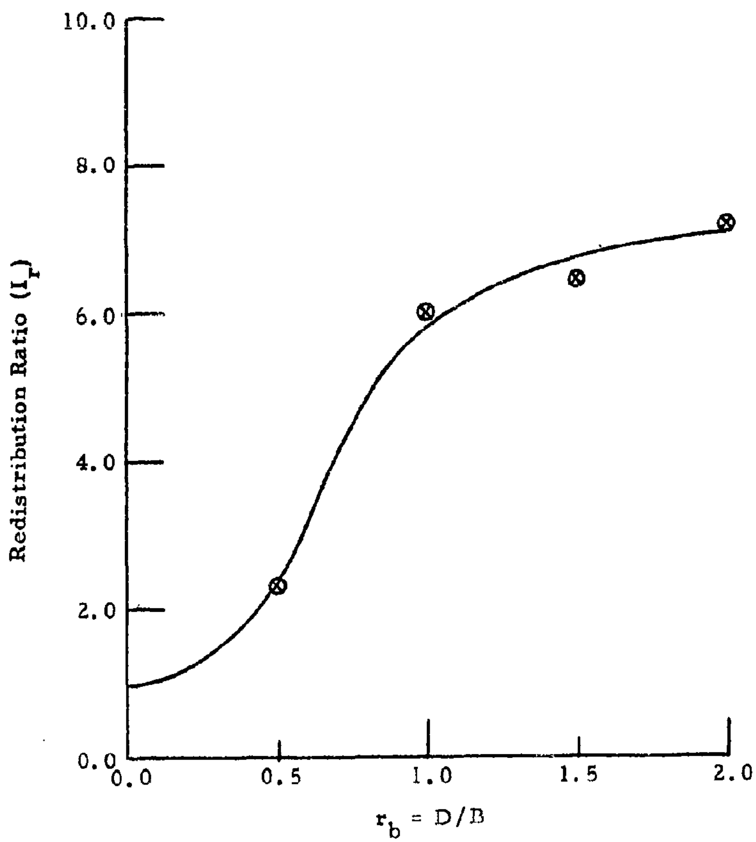


Fig. A-26 STRUCTURAL LOAD REDISTRIBUTION RATIO AS RELATED TO BURIAL RATIO, THREE DIMENSIONAL STATIC TESTS, MEDIUM DENSE SAND, r_Δ = 0.050

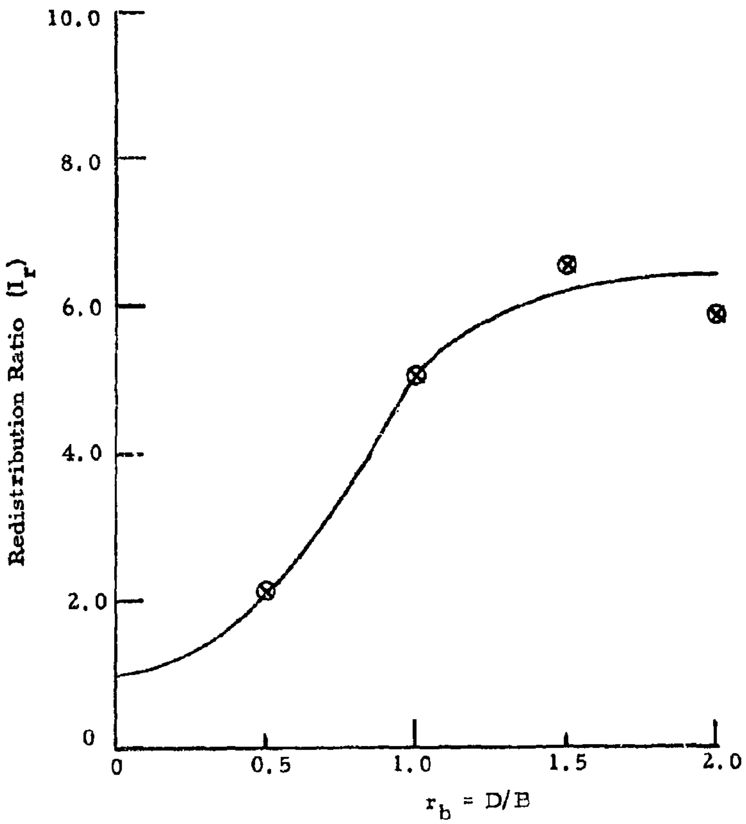


Fig. A-27 STRUCTURAL LOAD REDISTRIBUTION RATIO AS RELATED TO BURIAL RATIO, THREE DIMENSIONAL STATIC TESTS, MEDIUM DENSE SAND, $r_\Delta = 0.075$

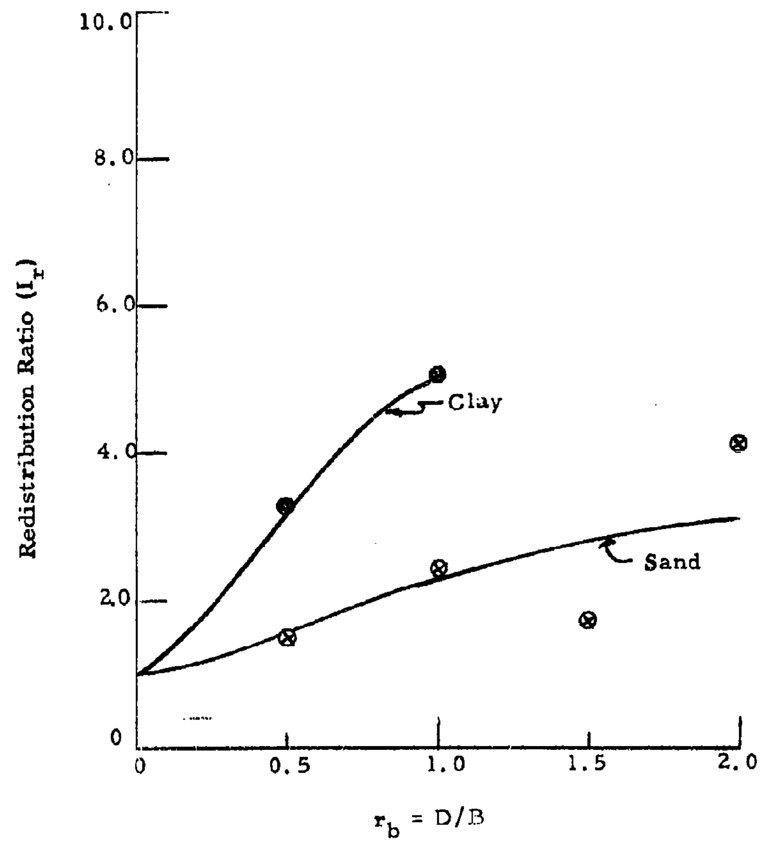


Fig. A-28 STRUCTURAL LOAD REDISTRIBUTION RATIO AS RELATED TO BURIAL RATIO, TWO DIMENSIONAL DYNAMIC TESTS

APPENDIX B

FIBER REINFORCED PLASTIC SHELTERS

by

Ralph L. Barnett

APPENDIX B

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FIBER REINFORCED PLASTIC SHELTERS

FIBER REINFORCED PLASTIC SHELTERS

I. INTRODUCTION

The feasibility of using plastics for underground shelters was established in 1959 in an investigation for the U. S. Navy Civil Engineering Laboratory[1]. At that time, it was found that fiberglass reinforced plastic (FRP) was the only plastic material suitable for structural applications involving large components. This material is formed by laminating glass fibers in a resin matrix. The possible combinations of resin-glass systems is extremely large due to the variety of available resins, glass systems, and fabrication techniques. After commenting very briefly on a few system components, we shall sharply focus our attention on the relative characteristics of one particular type of laminate; namely, the spray-up polyester laminate.

Polyester and epoxy resins are the most common types of laminating resins. The polyester systems are generally easier to control during fabrication and are less expensive than the epoxy systems. However, the epoxy formulations generally provide a stronger material, particularly when water conditions exist. Polyester based laminates lose approximately 20 percent of their ultimate strength in water while epoxies lose about 10 percent. It is common practice in the small boat industry to use the cheaper polyester resin and increase the wall thickness to compensate for the loss in strength. In other respects, the polyester and epoxy systems are quite similar.

Three basic fiberglass raw material forms are currently used in FRP construction; cloth, woven roving, and chopped fiber mat. Some of the advantages and disadvantages of these reinforcement types are given in Table I. We are particularly interested in the chopped mat fiberglass because of its low cost.

The following typical fabrication methods are used in the manufacture of structural plastics.

1. Matched Metal Molding

Matched metal molding involves the use of two mated molds mounted

Table B-1

COMPARATIVE ADVANTAGES AND DISADVANTAGES
OF THE THREE POPULAR REINFORCEMENT TYPES

	Advantage	Disadvantage
MAT OR CHOPPED FIBER	1. Low cost 2. Equal strength in all directions 3. Good interlaminar bonding 4. High bulk for building thickness 5. Forms to contours without bridging 6. Good strengths when used under pressure	1. Limited glass loading under contact pressure 2. Somewhat weaker than fabrics 3. Less uniform than cloth 4. Poor surface finish on non-mold surface 5. Thickness control difficult without pressure
CLOTH	1. High strength (in direction of strands) 2. High uniformity of glass distribution 3. Uniformity of finish "on non-mold surface" 4. Uniformity of laminate thickness 5. High glass loading	1. Highest cost 2. Slow build up for thickness 3. Problem of interlaminar shear 4. High strength only in direction of strands 5. Bridges in sharp radii
WOVEN ROVING	1. Properties and advantages of cloth at a price closer to mat 2. Highest impact strength 3. Good bulk for building thickness	1. Disadvantages of cloth 2. Tendency of weave pattern to telegraph or show through 3. Compacted strands difficult to saturate 4. Resin-rich interstice

in a press. Fiberglass is placed at the desired locations of the mold, and resin poured over the fabric. The press is then closed to provide pressure, while heat is applied. This type of manufacture generally results in the best quality laminate. However, the press tends to damage the fabric when used in complicated shapes such as a corrugation. For this reason such complicated shapes would result in higher unit costs than smooth surfaces. Presses for this type of molding generally do not exceed 40 square feet.

2. Vacuum Bag Molding

Vacuum bag molding consists of a single mold of wood, plaster, or metal. Glass and resin are applied to the mold, and a rubber bag conforming to the shape of the mold is placed over the material. A vacuum is then drawn between mold and bag, and the resin is manually worked through the fabric. The vacuum helps to draw the resin through the glass, and tends to remove the entrapped air from the laminate. This method also yields a sound laminate, but is limited to panels having an area of approximately 30 square feet or less.

3. Rubber Bag Molding

Rubber bag molding is similar to vacuum bag molding, but the bag is pressurized; therefore, positive pressure is applied to the laminate.

4. Autoclave Process

The autoclave process is used to supplement the vacuum bag technique. In this method, the formed, wetted laminate, and vacuum bag are placed in an autoclave chamber, which is steam heated and pressurized. This yields a laminate which is of good quality since substantial pressures and high temperatures may be achieved. Autoclaves have been frequently employed in the fabrication of radome panels.

5. Open-Hand Layup

This technique utilizes a mold upon which the resin and fabric are placed. The entire mold is then covered with plastic film, and the resin is worked through the fabric by hand. When the glass has been properly "wet out," the mold is set aside until curing of the resin is complete. This curing may be done at room temperature, or heat may be applied with heat lamps

or other means. This method does not yield laminates of the quality of the other methods, and results in an extremely slow rate of production. This technique permits larger panel sizes than the other methods, but also costs as much as 50 percent more than the matched molding method when large numbers of components are manufactured. For small quantities of large parts, the low mold cost makes this method more economical than the matched molding procedure.

6. Spray Gun

In this method, a spray gun is fitted with a chopper in such a way that short (3/4") lengths of fiber can be deposited simultaneously with a resin. The chopped fibers form a mat of fibers with random orientations. The resulting laminates are subject to extreme thickness variations about some nominal thickness; however, the ratio of glass to resin at any point in the laminate varies only slightly. As in the case of hand-lay-up, the mold costs are quite low and the possible sizes and thicknesses of components are unlimited. Finally, the cost per unit weight of large and small components of either simple or reasonably complex shapes is amazingly constant when surface finish is unimportant. The structural capabilities of the materials formed by this process have been proven through their use in boat hulls, automobile and truck bodies, and in large storage tanks and bins.

II. CHARACTERISTICS OF SPRAY-UP POLYESTER LAMINATES

A. General Advantages

In this subsection we shall indicate a number of advantages which are generally attributed to FRP Spray-Up structures.

1. Corrosion Resistance

2. Water-proofing

The versatility of the spray-up method enables one to produce large monolithic structures and thereby avoid the troublesome water-proofing of many joints. In 1960, an excellent report was issued by Rome Air Development Center on spray-in-place shelters[2]. This report establishes the feasibility of fabricating large monolithic FRP structures in the field. In addition, it contains a detailed description of the spray-up process.

3. Variable Thickness Control

The design of minimum weight structures generally calls for plates and shells of variable thickness. By properly establishing the "shooting schedules" during spray-up, it is relatively easy to control the thickness throughout a structure. By contrast, continuous thickness variations in metal shells can be effected only at great cost.

4. Easy Repair

5. Easy-Cut-Thru

Although seldom pointed out, the debris which attends a nuclear detonation has a high likelihood of blocking the entrance systems of shelters. Furthermore, if pressure levels are experienced which are somewhat higher than anticipated, it is possible that the functioning of blast doors may be impaired. In the case of such eventualities, it might be desirable to cut through various parts of the structure as a means of egress. This is accomplished in steel and concrete structure only with special tools and much labor. In FRP however, the simplest hand saw can be used to quickly cut through any surface.

6. Scarcity of Conventional Materials

It has been pointed out in Reference 1 that the seriousness of a wartime scarcity of conventional building materials must be kept in mind when comparing the relative desirability of FRP, steel, and reinforced concrete. It should be remembered that increasing use of FRP for military purposes may also make this material somewhat scarce in a war emergency.

B. General Characteristics

For all practical purposes, the tensile load on an FRP laminate is carried by the glass fibers. Consequently, one usually strives to achieve a high glass content laminate. In the spray process it becomes extremely difficult to wet out the glass fibers when the glass content exceeds 50 percent by weight. Minimum cost laminates generally use from 30 to 38 percent of glass. At these percentages, the compressive strength of the laminates is greater than their tensile strength. In high performance laminates (high glass content) one finds the opposite effect, e.g., here the bending specimens

fail on the compressive side. The nature of the glass matrix in most FRP laminates makes the propagation of cracks extremely difficult. As a consequence, catastrophic failures are seldom found in spray-up FRP structures.

It appears that each glass fiber in a laminate provides resistance only in the direction parallel to its length. The presence of forces perpendicular to the fibers do not affect the primary resistance parallel to the fiber. The strength of a laminate in a given direction is then proportional to the total projected cross-sectional area in that direction and is independent of the forces acting in other directions. On this basis we can predict strength in a biaxial stress field using a maximum stress theory. For isotropic laminates, such as a spray-up laminate, we then have a simple strength criterion.

III. COMPARISON OF FRP WITH CONVENTIONAL MATERIALS

A. Weight Comparisons

There are certain cost advantages which accrue to light weight structures by virtue of their lower shipping and erection costs. For this reason it appears useful to describe the relative efficiency of FRP spray-up laminates with reference to conventional materials.

The selection of materials for minimum weight applications is generally accomplished through the use of merit indices[3]. These indices are used to compare FRP with steel and aluminum in Table II. In each of the cases tabulated, the structural weight is inversely proportional to the merit index. We may conclude from Table II that FRP is inferior to steel and aluminum in both stiffness and stability applications; it is superior in elastic impact. In strength applications, FRP is much better than steel and about equal to aluminum.

B. Cost Comparisons

The considerations of the previous section on weight are only incidental to the real problem of shelter design, - minimum cost. In this section our attention is turned to cost comparisons of the most elemental structural components; namely, the plate in tension and flexure and the sandwich panel in flexure. In Table III, the cost of FRP members is compared to equivalent members in mild steel. Such a comparison reflects only the gross features entering into the total costs of either material, and for this reason, the cost

Table B-2

MERIT INDICES

Structural Application	Index	FRP	Mild Steel	Aluminum 6061-T6
Tension	σ_t/ρ	239×10^3 in	212×10^3 in	388×10^3 in
Bending	σ/ρ	376×10^3 in (ultimate)	117×10^3 in (yield)	357×10^3 in (yield)
Compression	σ_c/ρ	367×10^3 in	117×10^3 in	357×10^3 in
Buckling Solid Column	$E^{1/2}/\rho$	14×10^3	19.3×10^3	32.5×10^3
Buckling Thin-Walled Circular Tube	$E^{2/3}/\rho$	12.9×10^4	34.1×10^4	47.4×10^4
Elastic Impact	$\frac{(\sigma/\rho)^2}{(E/\rho)}$	129×10^2 in	1.37×10^2 in	12.7×10^2 in
Stiffness	E/ρ	11×10^6 in	100×10^6 in	100×10^6 in

Physical Properties of FRP

Tensile Strength	13,000 psi
Compressive Strength	20,400 psi
Flexural Strength	20,900 psi
Modulus of Elasticity (tension)	0.23×10^6 psi
Modulus of Elasticity (flexure)	0.61×10^6 psi
Weight Density	0.0555 lbs/in^3
Glass Content (by weight)	30 percent
Modulus of Toughness	$\sim$ twice modulus of resilience

Table B-3

COST COMPARISONS OF FRP AND MILD STEEL

Plate in Tension - Equivalent Strength Sandwich Panel in Flexure - Equivalent Strength	$(Cost)_o = \left(\frac{\$^o \sigma^s}{\$^s \sigma^o}\right) (Cost)_s$ $(Cost)_{FRP} = 3.42\ (Cost)_{steel}$
Plate in Tension - Equivalent Stiffness Sandwich Panel in Flexure - Equivalent Stiffness	$(Cost)_o = \left(\frac{\$^o E^s}{\$^s E^o}\right) (Cost)_s$ $(Cost)_{FRP} = 65.7\ (Cost)_{steel}$
Plate in Tension - Equivalent Elastic Impact Resistance Sandwich Panel in Flexure - Equi. Elastic Impact Resistance	$(Cost)_o = \left(\frac{\$^o E^o}{\$^s E^s}\right)\left(\frac{\sigma^s}{\sigma^o}\right)^2 (Cost)_s$ $(Cost)_{FRP} = 0.178\ (Cost)_{steel}$
Plate in Flexure - Equivalent Strength	$(Cost)_o = \left(\$^o/\$^s\right) \sqrt{\sigma^s\ \sigma^o}\ (Cost)_s$ $(Cost)_{FRP} = 1.71\ (Cost)_{steel}$
Plate in Flexure - Equivalent Stiffness	$(Cost)_o = \left(\$^o/\$^s\right) \left(E^s/E^o\right)^{1/3} (Cost)_s$ $(Cost)_{FRP} = 4.90\ (Cost)_{steel}$
Plate in Flexure - Equi. Elastic Impact Resistance	$(Cost)_o = \left(\$^o E^o/\$^s E^s\right) \left(\sigma^s/\sigma^o\right)^2 (Cost)_o$ $(Cost)_{FRP} = 0.072\ (Cost)_{steel}$
Plate in Tension - Equi. Plastic Impact Resistance Sandwich Panel in Flexure - Equi. Plastic Impact Resistance	$(Cost)_o = \left(\frac{\$^o}{\$^s}\right)\frac{(Mod.\ Tough.)^s}{(Mod.\ Tough.)^o} (Cost)_s$ $(Cost)_{FRP} = 104.5\ (Cost)_{steel}$

FRP: $\$^o = 0.05667\ \$/in^3$; Steel: $\$^s = 0.04245\ \$/in^3$

relationships can be quite different in special situations.

In preparing Table III, all compromises were made in favor of the FRP. The unit cost for the FRP laminate was taken as $1.02 per pound which is somewhat lower than usually quoted. It is assumed that we have a filled resin, mass production, and that no great concern be given to surface finish. The unit cost of the steel plate in place was taken at $0.15 per pound.

Examination of Table III indicates that FRP is more costly than steel in every application except elastic impact. The closest comparison is found to be in the bending strength of solid laminates. It is very clear that FRP structures are competitive only in special applications where several of their desirable characteristics can be combined. Since shell structures may be just such special cases, a few remarks will be made about these structures in the concluding sections of this appendix.

C. Selection of Panels

Plastic and FRP laminates are available in an enormous variety of forms and it is customary to use some measure of unit stress to evaluate their strength. Unfortunately, this evaluation procedure is strictly valid only for homogeneous specimens whose stress distribution can be found from equilibrium conditions alone, e.g., tension specimens, thin-walled circular torsion specimens, or thin-walled cylinders under internal pressure. For non-homogeneous members, such as sandwich panels, a single number for the ultimate stress is without meaning. Furthermore, when the stress-strain characteristics of a material are non-linear, as in the case of some thin spray-up laminates, utilization of the modulus of rupture $\left(\sigma = \frac{M_{ult.}}{I/c}\right)$ can lead to very large errors in the predicted moment capacity of different thickness laminates.

At the present time, it would seem to be more reasonable to report the moment capacity per unit width for every laminate rather than their modulus of rupture. With this information available, it is the possible to construct a Cost-Moment Capacity diagram which could include all types of panels regardless of material, non-linearity, or non-homogeneity. A moment-thickness diagram for spray-up laminates is presented in Fig. 1. If this

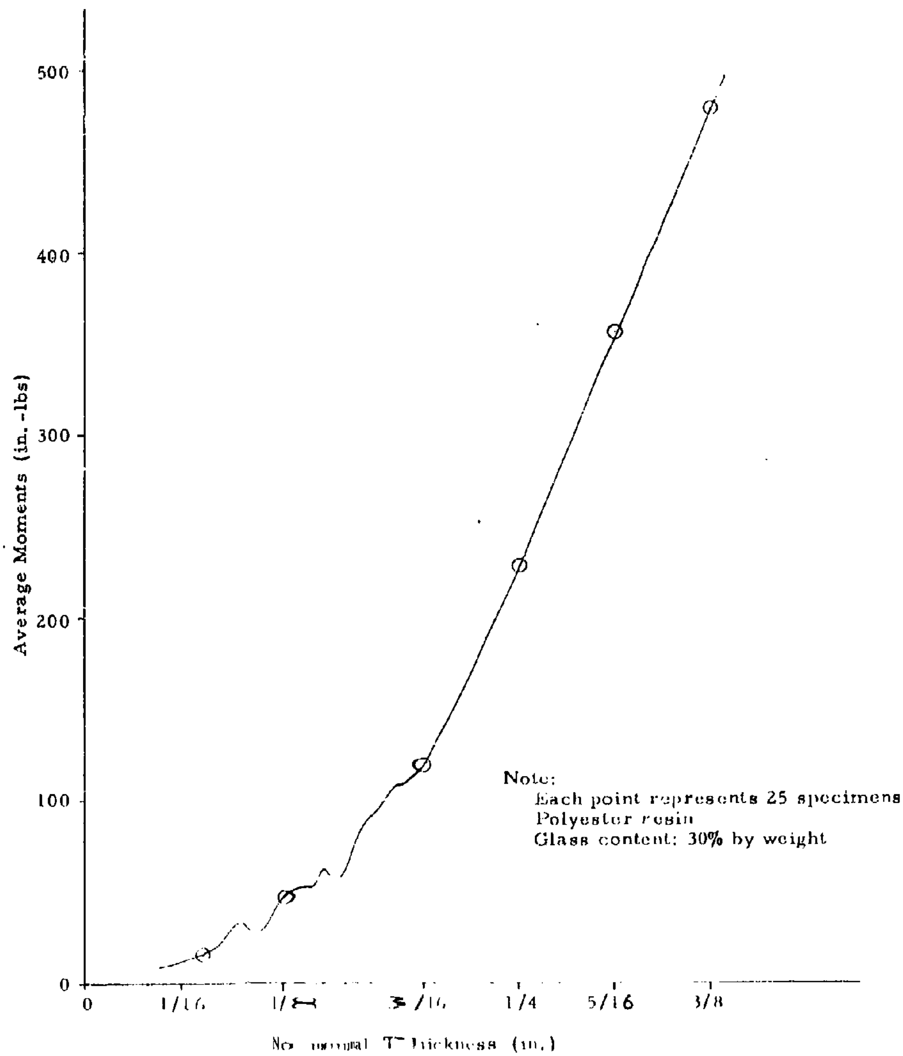

Figure B-1

MOMENT CAPACITY-- THICKNESS RELATIONSHIP FOR SPRAY-UP LAMINATE

curve were based on the modulus of rupture of the 1/16 inch laminate, the moment capacity for the 3/8 inch laminate would be in error by about 10 percent.

IV. SHELL STRUCTURES

A. General Remarks

In our previous comparisons of FRP and steel components, we found that steel at $300 per ton produced lower cost elements. For shell structures which have non-developable geometries, it is generally necessary to press flat steel plates into segments of a shell and weld these segments into a complete shell. The labor involved in such fabrication is high and the amortized die cost for small numbers of special shapes is enormous. As we have already indicated, the cost of relatively intricate shapes in FRP is just about the same as for simple shapes; therefore, it is quite possible for FRP to be competitive with non-developable steel shells.

In the remaining two sections we shall consider several characteristics of the design of spherical caps and hyperbolic paraboloids.

B. Spherical Caps

Consider the design of the hydrostatically loaded spherical cap shown in Fig. 2. According to the maximum stress theory this shell will fail when the maximum principal stress is equal to the tensile strength of the material σ_T. To resist a pressure p, the smallest thickness must be taken as

$$t = \frac{pr}{2\sigma_T} \tag{1}$$

Using this thickness, we find the weight of the shell to be

$$W = At\rho = \frac{p\pi r^2 h}{(\sigma_T/\rho)} \tag{2}$$

where h is the height of the cap as shown in Fig 2. From the geometry of the cap,

$$r = \frac{h^2 + k^2}{2h} \tag{3}$$

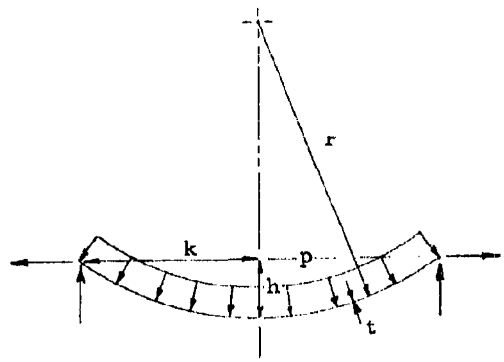

Figure B-2

HYDROSTATICALLY LOADED SPHERICAL CAP

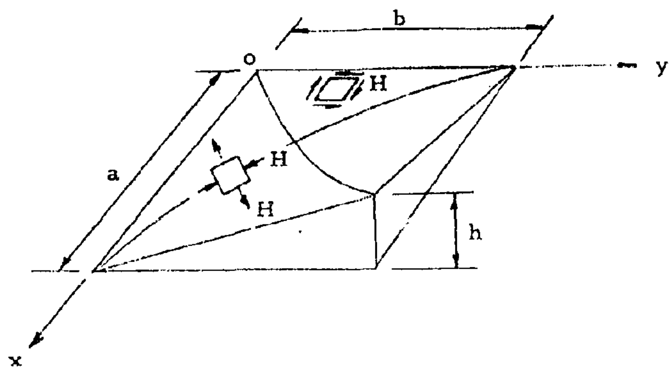

Figure B-3

HYPERBOLIC PARABOLOID UNDER A UNIFORM VERTICAL PRESSURE

Thus, the shell weight per unit of base area is

$$\frac{W}{\pi k^2} = \frac{p\ k}{4\left(\sigma_T/\rho\right)} \frac{\left[1 + (h/k)^2\right]^2}{(h/k)} \tag{4}$$

It is clear from this equation that for similar shells the weight per unit enclosed area increases linearly with the span. To find the minimum weight shell for a given span 2k, we differentiate $W/\pi k^2$ with respect to (h/k) and set the result equal to zero; thus, the optimum ratio becomes

$$(h/k)_o = 1/\sqrt{3} \quad \text{or} \quad r_o = k2/\sqrt{3} \tag{5}$$

and the corresponding minimum weight is

$$\left(\frac{W}{\pi k^2}\right)_o = \frac{4\sqrt{3}}{9} \frac{p\ k}{\left(\sigma_T/\rho\right)} \tag{6}$$

Clearly, the weight varies inversely with the specific tenacity, $\left(\sigma_T/\rho\right)$. As a matter of interest, the hemispherical shell weighs 30 percent more than the optimum shell. When a shell is evaluated together with the other components of an entire structure, the optimum radius will generally be different from that described here.(4) However, the main features of the optimum design are reflected by this treatment.

C. Hyperbolic Paraboloids

The variety of roof systems which can be synthesized using hyperbolic paraboloid (HP) panels is quite extensive; however, it is possible to study a wide range of such structures by considering the weight per unit projected area of a typical panel. The equation of the HP surface shown in Fig. 3 is

$$z = \frac{h}{ab} xy \tag{7}$$

When the slopes on this surface are small, one obtains a simple expression for the shell area:

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When the slopes on this surface are small, one obtains a simple expression for the shell area:

$$A = \iint_S \left[1 + \left(\frac{\partial z}{\partial x}\right)^2 + \left(\frac{\partial z}{\partial y}\right)^2\right]^{1/2} dy dx$$

$$\doteq \int_0^a \int_0^b \left[1 + \frac{h^2}{2a^2 b^2} \left(y^2 + x^2\right)\right] dy\, dx \tag{8}$$

or

$$A \doteq ab\left[1 + \frac{h^2}{6}\left(\frac{a^2+b^2}{a^2 b^2}\right)\right] \tag{9}$$

The membrane stresses in the HP shell under a uniform vertical pressure p form a homogeneous state of pure shear which can be simply exprcssed as

$$H = \frac{pab}{2h} \tag{10}$$

where H is the shearing force per unit width of shell. Assuming that the strength of an FRP laminate is governed by the maximum tensile stress, the required thickness is simply $t = H/\sigma_T$. Using the expressions for t, H, and A, the shell weight per unit projected area becomes

$$\frac{W}{ab} = \frac{pa}{2\left(\sigma_T/\rho\right)}\left[\frac{1}{(h/b)} + (h/b)\left(\frac{a^2+b^2}{6a^2}\right)\right] . \tag{11}$$

We observe that this weight expression is similar to that of the spherical cap, i.e., the weight is inversely proportional to the specific tensile strength and for similar geometries the weight is proportional to the span a.

We may proceed formally to obtain the optimum HP proportions by setting

$$\frac{d(W/ab)}{d(h/b)} = 0 \tag{12}$$

from which it follows that

$$(h/b)_o = a\sqrt{\frac{6}{a^2+b^2}} \tag{13}$$

The corresponding optimum weight becomes

$$\left(\frac{W}{ab}\right)_o = \frac{p}{|\sigma_T/\rho|}\sqrt{\frac{a^2+b^2}{6}} \tag{14}$$

Unfortunately, this optimum shell extends the shell dimensions beyond their region of applicability due to the assumption of small slope employed when computing the area. We note, for example, that when a = b, $(h/a)_o = \sqrt{3}$ which is far too large for the area approximation. For slopes less than unity the approximation is quite reasonable and Eq.(11) can be expected to reflect quite closely the weight per unit projected area:

In spite of its shortcomings, the optimization of the HP shell provides two useful results. First, to minimize the shell weight the HP should be designed with the biggest possible rise consistant with excavation cost. One intuitively feels that rises in the order of four times the span a will never be economical. The second result is simply the observation that the approximation for the area is always too high which implies that the optimum weight given by Eq.(14)is too large.

Our closing remarks shall be directed toward the comparison between the spherical cap and the HP shell. The optimum height of the spherical cap is 1/3 that of the HP which strongly favors the cap. However, even with the exaggerated prediction for the optimum weight of the HP, this shell is superior to the spherical cap, i.e.,

$$\left(\frac{W_{CAP}}{\pi k^2}\right)_o \doteq \frac{4}{3}\left(\frac{W_{HP}}{a^2}\right)_o \qquad a = b = k \tag{15}$$

Perhaps a more useful comparison can be made by requiring that both the span and the rise be kept equal; thus

$$\left(\frac{W_{CAP}}{\pi k^2}\right) \Big/ \left(\frac{W_{HP}}{a^2}\right) = \frac{\left[1 + (h/k)^2\right]^2}{2\left[1 + \frac{1}{3}(h/k)^2\right]} \qquad (16)$$

When this relationship is studied it is apparent that the spherical cap is superior for values of (h/k) below <u>0.73</u>. For greater values of (h/k) the HP weighs less than the cap.

REFERENCES

lackett, D. A., "Feasibility Study Concerning the Utilization of lastics for Underground Personnel Shelters," Navy Dept., Contract). NBy - 3150, ASTIA No. AD228706, Sept. 1959.

rdo, A. F., Schramp, J. M., McCormick, J. E., and Stabler, G. "Spray-In-Place Shelters," Rome Air Development Center, Report RADC-TN-60-40, ASTIA No. AD-234867, March 1960.

4. ett, R. L., "Selection of Material in Minimum Weight Design," Design Engineering Conference, ASME, May 1963.

rt, R. L., "Comparative Protective Structural Design, Hardening eapons Systems," Air Force Special Weapons Center, Contract 29(601) - 547, ARF Project 8172, 1960, pp. IV-17, Confidential.

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